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NATIONAL DAM SAFETY PROGRAM. CORK CENTER STORAGE RESERVOIR DAM --ETC(U)
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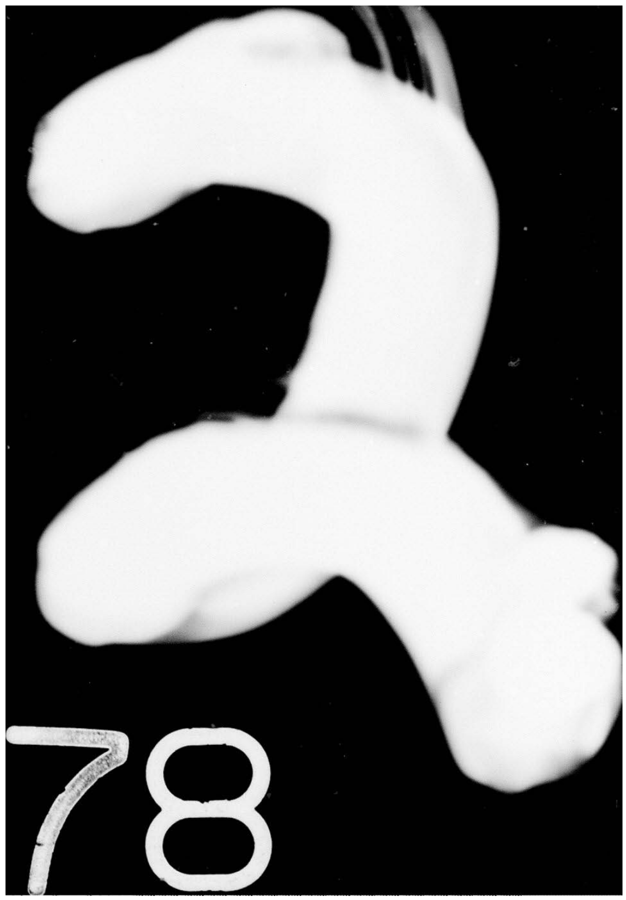
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LEVEL II

MOHAWK RIVER WATERSHED
KECKS CENTER CREEK BASIN

6

CORK CENTER STORAGE RESERVOIR DAM
FULTON COUNTY, NEW YORK

NY 658
PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

Prepared by
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CONSULTING ENGINEERS
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CALDWELL, NEW JERSEY 07006

For

DEPARTMENT OF THE ARMY
NEW YORK DISTRICT, CORPS OF ENGINEERS
26 FEDERAL PLAZA
NEW YORK, NEW YORK 10007

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DEPARTMENT OF THE ARMY
U. S. ARMY ENGINEER DISTRICT, NEW YORK
26 FEDERAL PLAZA
NEW YORK, NEW YORK 10007

2 OCT 1978

NANEN-F

Honorable Hugh L. Carey
Governor of New York
Albany, New York 12224

Dear Governor Carey:

The purpose of this letter is to inform you of a clarification of the guidelines used by this office in assessing dams under the National Program of Inspection of Dams.

Office of the Chief of Engineers has recently provided a clarification that dams with seriously inadequate spillways are to be assessed as unsafe, non-emergency, until more detailed studies prove otherwise or corrective measures are completed.

The following dams in your state have previously been assessed as having seriously inadequate spillways, with capability to pass safely only the percentage of the probable maximum flood as noted in each report. They are now to be assessed as unsafe:

I.D. NO.	NAME OF DAM
N.Y. 59	Lower Warwick Reservoir Dam
N.Y. 4	Salisbury Mills Dam
N.Y. 45	Amawalk Dam
N.Y. 418	Jamesville Dam
N.Y. 685	Colliersville Dam
N.Y. 6	Delta Dam
N.Y. 121	Oneida City Dam
N.Y. 39	Croton Falls Dam
N.Y. 509	Chadwick Dam (Plattenkill)
N.Y. 66	Boyds Corner Dam
N.Y. 397	Cranberry Lake Dam
N.Y. 708	Seneca Falls Dam
N.Y. 332	Lake Sebago Dam
N.Y. 338	Indian Brook Dam
N.Y. 33	Lower(S) Wiccopee Dam (Lower Hudson W.S. for Peekskill)

NANEN-F
Honorable Hugh L. Carey

I.D. NO.	NAME OF DAM
N.Y. 49	Pocantico Dam
N.Y. 445	Attica Dam
N.Y. 658	Cork Center Dam
N.Y. 153	Jackson Creek Dam
N.Y. 172	Lake Algonquin Dam
N.Y. 318	Sixth Lake Dam
N.Y. 13	Butlet Storage Dam
N.Y. 90	Putnam Lake (Bog Brook Dam)
N.Y. 166	Pecks Lake Dam
N.Y. 674	Bradford Dam
N.Y. 75	Sturgeon Pool Dam
N.Y. 414	Skaneateles Dam
N.Y. 155	Indian Lake Dam
N.Y. 472	Newton Falls Dam
N.Y. 362	Buckhorn Lake Dam

The classification of "unsafe" applied to a dam because of a seriously inadequate spillway is not meant to connote the same degree of emergency as would be associated with an "unsafe" classification applied for a structural deficiency. It does mean, however, that based on an initial screening, and preliminary computations, there appears to be a serious deficiency in spillway capacity so that if a severe storm were to occur, overtopping and failure of the dam would take place, significantly increasing the hazard to loss of life downstream from the dam.

Consequently, it is advisable to implement the recommendations previously furnished in the reports for the above-mentioned dams as soon as practicable.

It is requested that owners of these dams be furnished a copy of this letter and that copies be permanently appended to all reports previously furnished to you.

Sincerely yours,

CLARK H. BENN
Colonel, Corps of Engineers
District Engineer

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

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3. RECIPIENT'S CATALOG NUMBER

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Inventory No. N.Y. 658

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16 Gary S. Salzman P.E.

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Kecks Center Creek
Cork Center Storage Reservoir
Fulton County

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report provides information and analysis on the physical condition of the dam as of the report date. Information and analysis are based on visual inspection of the dam by the performing organization.

Cork Center Reservoir Dam was judged to be unsafe-non-emergency due to a seriously inadequate spillway.

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UNCLASSIFIED

MOHAWK RIVER WATERSHED
KECKS CENTER CREEK BASIN
FULTON COUNTY, NEW YORK

CORK CENTER STORAGE RESERVOIR DAM

CITY OF JOHNSTOWN, NEW YORK
DEPARTMENT OF WATER
NDS # NY 658,
NYSDEC # 172C-3191

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

Prepared by

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For

DEPARTMENT OF THE ARMY
New York District, Corps of Engineers
26 Federal Plaza
New York, New York 10007

29 August 1978

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PHASE I REPORT

NATIONAL DAM SAFETY PROGRAM

BRIEF ASSESSMENT OF GENERAL CONDITIONS

AND

RECOMMENDED ACTION

Name of Dam: Cork Center Storage Reservoir Dam

Owner: City of Johnstown, N.Y.; Department of Water

State Located: New York

County Located: Fulton

Stream: Keck Center Creek

Date of Inspection: 20 July 1978

Inspection Team: Converse Ward Davis Dixon
91 Roseland Avenue, P. O. Box 91
Caldwell, New Jersey 07006

Based on our visual inspection, a review of available data, and calculations performed as part of this study, the Cork Center Storage Reservoir Dam is judged to be functioning satisfactorily at this time. However, based on the screening guidelines established by the Department of Army, Office of the Chief of Engineers (OCE), the spillway capacity is rated as inadequate. In addition, the spillway is seriously inadequate since it satisfies all the conditions established by the OCE guidelines for determining seriously inadequate spillway capacity. Since this assessment was based on OCE screening criteria, a detailed hydrologic and hydraulic evaluation of the watershed and spillway should be performed by the use of more precise and sophisticated methods and procedures. Following such an investigation, the need for, and type of, mitigating measures should be determined. Until such a study is completed and the spillway adequacy issue resolved, around-the-clock surveillance of the dam should be provided during periods of unusually heavy precipitation.

The occurrence of excessive amounts of seepage along the toe of the embankment warrants further investigation. It is recommended that the nature, strength properties and seepage characteristics of the embankment and foundation materials be established through a soil exploratory and testing program. The stability of the embankment may then be analyzed in the light of the new findings, and any necessary measures to reduce or control flow established.

Our assessment of the general physical condition of the Cork Center Storage Reservoir Dam has led us to make the following recommendations which should be implemented as soon as practicable, certainly within the next three years:

1. Injection grouting for fixing the leak between the downstream slope of the overflow spillway and the left abutment should be undertaken.

2. All minor damages to concrete (spalling, scaling, etc.) should be repaired.

3. The intake structure access bridge should be scraped and subsequently painted.

4. The gate house should be lighted.

5. The low woody growth on the upstream face of the dam should be removed. Shallow rooted trees on the embankment should be cut down; deep rooted trees should remain.

6. A specific program of periodic maintenance of the dam embankment and its appurtenant structures should be established and followed.

7. The flow from the lake should be controlled from the intake structure rather than from the lower gate house.

Respectfully submitted,

CONVERSE WARD DAVIS DIXON

Gary S. Salzman

Gary S. Salzman, P.E.

Date: 29 August 1978

Approved by: Clark H. Benn

Colonel Clark H. Benn
New York District Engineer

Date: 21 September 78

OVERVIEW - CORK CENTER STORAGE RESERVOIR

ABSTRACT

SECTION 1

PROJECT INFORMATION

1.1 General

a. Authority

The authority to conduct this Phase I inspection and evaluation comes from the National Dam Inspection Act (P.L. 92-367) of 1972 in which the Secretary of the Army was authorized to initiate, through the Corps of Engineers, a program of safety inspections of non-federal dams throughout the United States. Management and execution of the program within the State of New York has been undertaken by the New York State Department of Environmental Conservation (NYSDEC).

b. Purpose

The primary purpose of the inspection is to evaluate available data and to give an opinion as to whether the subject dam constitutes a hazard to human life or property.

ABSTRACT

1.2 Description of Project

a. Description of Dam and Appurtenances

The Cork Center Storage Reservoir was built in 1919, and is an earth fill structure with a concrete core wall (Plates IV and V). It is approximately 460 feet in length along its crest and 49 feet high at its deepest section (near the middle). Longitudinally, the earth embankment generally lies in the north-south direction with its crest at elevation 1059. The upstream and downstream faces slope at approximately 1 vertical to 3 horizontal, and 1 vertical to 2 horizontal, respectively. An unpaved access road extends over the crest, which is 10 feet wide.

Based on the design drawings, the top of the concrete core wall is 2 feet wide and is located 2 feet below the crest of the dam along its centerline. It tapers at 1 horizontal to approximately 19 vertical on both its upstream and downstream sides, and its height varies with the depth of embankment above the original ground surface. Its maximum base width is 6 feet. Near the right end of the embankment where the dam height is small, the concrete core wall is of uniform thickness, being 3 feet from top

to bottom. The depth of core wall penetration below the original ground surface varies from about 7 feet to 12 feet, and it is embedded into rock, roughly between Stations 0+70 and 2+90 (Plate V). The main purpose of the concrete core wall is to inhibit seepage through the embankment and to reduce the amount of seepage under it. Plate V also shows that the rip-rap on the upstream face of the embankment extends from elevation 1057 down to original ground surface except where original surface is below elevation 1021, in which case rip-rap is carried to elevation 1021 only.

The outlet works consist of the following items:

i. A 500-foot long concrete-lined spillway, 60 feet wide at the overflow section.

ii. Two 24-inch C.I. pipes. One pipe is used for water supply to the Town of Johnstown. The second 24-inch C.I. mud pipe is available for emergency conditions.

The spillway overflow section is located at the right abutment of the earth embankment and in a direction perpendicular to it (Plate IV). It is a concrete gravity structure and ungated (Plate II). Wooden sheet piling was apparently installed below it (Plate IV). The 60-foot length of the overflow section is divided into three equal bays by two piers of 1-foot width each. These piers and the end abutments support a concrete footbridge over the spillway. The spillway crest was raised by 15 inches and its plans for reconstruction were approved by N.Y. State Department of Public Works on October 17, 1963. The rounded crest of the remodelled spillway is at elevation 1055.25 feet, which is 3.75 feet below the abutment seats.

Reconstruction included a 12-inch thick reinforced concrete slab and wall dowelled into the upstream slab and along the vertical upstream face of the spillway, and raising the crest level by 15 inches (Plate II). The downstream face slopes at approximately 1 horizontal to 1.33 vertical and meets the downstream slab, also raised by 1 foot, at elevation 1048.0 feet. The spillway discharge channel (Plates IV and VI), constructed of concrete side-retaining walls and bottom concrete slab, takes two sharply curved turns and empties into the natural streambed about 150 feet downstream of the toe of the deepest section of the earthen embankment.

The intake structure as-built is not that shown on the original design drawings (Plate IV). The intake structure actually built in 1919, as shown in a photograph at the Water Department offices, consists of a vertical concrete cylindrical shell with an 11-foot inside diameter and 6-inch thick wall. Its top is at elevation 1060 and its vertical axis is about 98 feet upstream from the centerline of the crest of the embankment. The approach to the intake structure is a steel angle iron bridge built in 1963. Two valves located at 17-foot and 25-foot depths below the top of the intake (Fig. 69, Appendix E) let water into a 24-inch C.I. supply pipe, laid under the embankment, leading to a lower gate house located downstream of the dam, and then into the creek channel. There is a third valve at the intake structure for the 24-inch C.I. mud pipe, which is 40 feet below the top of intake (elevation 1020). The second 24-inch C.I. pipe (mud pipe) extends 30 feet farther from the intake structure into the lake, and its end is marked by a floating buoy. The mud pipe, which is used as an emergency outlet, follows the alignment of the supply pipe, namely under the embankment to the gate house and then to the natural drainage creek. The valves at the intake structure are kept open throughout the year, but they are worked once a year to check if the mechanism is functioning satisfactorily. Water supply is regulated by the 24-inch supply line valve in the lower gate house.

The gate house has a wooden floor, with stairs leading to the lower level for access to the gates. The gate house is not equipped with any lighting arrangement. In the gate house, there is one 24-inch gate valve across each of the two main lines (water supply line and mud line) and a 16-inch gate valve on the pipe that joins the two pipes (Fig. 70, Appendix E). In addition to the gate valve, there is a butterfly valve on the supply line.

Plates IV and V (1917 drawings) show a 24-inch valve at the embankment crest, but it was not encountered during the field inspection. Similarly, there is no aerator at the site, nor a 16-inch C.I. pipe bypass on the right side of the spillway, as shown on these drawings. It appears that during construction some changes were made from the original plans, deleting these items.

b. Location

The dam is locally called "High Daddy" and is located on Keck Center Creek in Fulton County, N.Y., approximately 5 miles northwest of the City of Johnstown, N.Y. The location of the dam is N43°02'14", W74°27'55",

shown on Plate I, which is a portion of the USGS 7.5 minute Quadrangle Sheet of Peck Lake, N.Y.

c. Size Classification

The dam is classified as intermediate (storage = 445 acre-feet; height = 49 feet).

d. Hazard Classification

This dam is classified as a "high hazard" due to the existence of danger to more than a few human lives (Miller farm, barn and house; trailer off Wemple Road near creek; two houses in Kecks Center).

e. Ownership

Department of Water
City of Johnstown
City Hall
Johnstown, New York

f. Purpose of Dam

The dam was built to act as a storage reservoir for the City of Johnstown water supply system. Its watershed is approximately 2.6 square miles. The flow from the 24-inch supply pipe empties into the natural streambed about 150 feet downstream of the toe of the embankment. The natural channel carries this water to a weir, where water is diverted into the purification plant. After treatment, the water is distributed in the City of Johnstown supply lines.

g. Design and Construction History

The dam was designed in 1917 by W. E. Natanson, the then Johnstown City Engineer, and James P. Wells, Consulting Engineer, Rochester, N.Y. It was approved by George D. Pratt, Commissioner, Conservation Commission, Division of Inland Waters, State of New York (Ref: back side of 1917 drawings 1 through 7; four of which are presented in this report as Plates III through VI).

Hydrologic computations and detailed stability analysis of the spillway section, performed in 1917, are presented in Appendix E. A summary of this analysis in the form of a memorandum dated July 24, 1917, from John Henry, the then junior engineer, N.Y. State Conservation Commission, to A. H. Perkins, division engineer, is also documented in Appendix E.

An application on Form E-61A1(2/62) dated Oct. 7, 1963, copy presented in Appendix E, was submitted by the City of Johnstown to the Bureau of Waterways, Division of Construction, Department of Public Works, Albany, N.Y., for raising the crest of the spillway by 15 inches to elevation 1055.25. One of the associated drawings prepared by Morrell Vrooman Engineers, Consulting Engineers, Gloversville, N.Y., is included in this report as Plate II. Another drawing in this set details the construction of a bridge from the embankment to the intake structure. The drawings were approved on Oct. 17, 1963.

During visual inspection, minor signs of pressure grouting or possibly just surface guniting were noticed around the spillway with a scratched date of 1963.

h. Normal Operational Procedure

The Department of Water, City of Johnstown, controls and regulates the water in the system. Water levels in the reservoir are checked twice a day for 7 days a week. Generally, in late fall, water is drawn down to approximately 1 foot below spillway crest and is maintained at that level through the winter. By April, water is purposely drawn down to about 6 feet below spillway crest so as to pass the spring and summer storms safely, with only a small amount overflowing the spillway. The reservoir level is brought down again in autumn. Usually water does not flow over the spillway; however, the highest water level reported is 2 feet overtopping the spillway (around 1971). Water from the Cork Center Reservoir is drawn through the intake structure and regulated by a gate valve on the 24-inch C.I. supply pipe at the lower gate house. Just downstream of the gate house, this water is discharged into the natural stream which carries it to a treatment facility. After treatment, water is supplied to the City according to the demand of the system.

The 24-inch C.I. mud pipe acts as an emergency outlet in case of danger of overtopping of the earthen embankment during a heavy storm. A copy of "Instructions for State of Emergency Conditions", hung on the office wall of the Department of Water in Johnstown, is attached to this report in Appendix E.

1.3 Pertinent Data

a. Drainage Area

The drainage area is approximately 2.6 square miles.

b. Discharge at Damsite

Maximum flood at the damsite is unknown, but it corresponds to about 2 feet of water over the spillway crest. This would result in a flow of approximately 645 cfs passing over the spillway.

Total spillway capacity at maximum pool elevation of 1059 feet is 1649 cfs.

c. Elevation (ft. above MSL)

Top of dam: 1059.

Maximum pool (top of dam): 1059.

Normal pool: Variable during different seasons; generally 1054 (approximate).

Overflow spillway crest: 1055.25.

Upstream supply pipe invert: 1043± and 1035±.

Downstream supply pipe invert: 1002±.

Upstream mud pipe invert: 1020±.

Downstream mud pipe invert: 1002±.

Streambed at supply and mud pipe outlet: 1002±.

d. Reservoir Length

Approximately 4000 feet from USGS Quad.

e. Storage (acre-feet)

Normal pool: Variable with seasons; generally 404 (estimated).

Spillway crest (El. 1055.25): 445.

Maximum pool (El. 1059): 601 (estimated).

f. Reservoir Surface

Normal pool: Variable in different seasons; generally 41 acres.

Spillway crest (El. 1055.25): 41 acres.

Maximum pool (El. 1059): 42.2 (estimated).

g. Dam

Type: Earth fill with concrete cutoff wall.

Length: Approximately 460 feet along crest.

Height: Variable; approximately 49 feet at greatest height near center.

Top width: 10 feet at crest.

Side slopes: Approximately 1 vertical to 3 horizontal upstream slope, and 1 vertical to 2 horizontal downstream slope.

Cutoff: Concrete cutoff wall, 2 feet wide at top and tapering upstream and downstream at 1 horizontal to approximately 19 vertical to a maximum width of 6 feet. Near the right end of the embankment, the cutoff wall is of uniform thickness of 3 feet from top to bottom. The wall is embedded below original ground surface for a minimum of 7 feet and a maximum of 12 feet, and penetrates to rock roughly between Stations 0+70 and 2+90 (Plate V).

h. Diversion and Regulating Works

Type: Two 24-inch diameter cast iron pipes at elevations 1043 and 1035 leading into the intake structure from whence only one 24-inch pipe proceeds to the lower gate house.

Length: 300 feet ± from intake structure to lower gate house, and 70 feet ± from lower gate house to discharge at stream.

Closure: Two manually operated valves at the intake structure for the supply line. Two manually operated valves at the lower gate house, either one of which can be used for control.

Access: To intake structure via bridge from embankment. Regulation from top of structure (intake box is locked with chain). At the gate house, regulation on the first floor of wood frame building (stairs to the lower level where valves are located).

Regulating facilities: Supply line valves at intake open at all times and regulation accomplished by 24-inch gate valve on the supply line at the lower gate house.

i. Spillway

Type: Concrete, round crested, gravity.

Length: 512 feet.

Width: 20 feet along channel, 60 feet at overflow section.

Crest elevation: 1055.25.

Gates: None.

Piers: 2; each 1 foot wide at the overflow section, carrying the bridge above.

j. Regulating Outlets (emergency)

Type: One 24-inch cast iron mud pipe at elevation 1020.

Length: 330 feet ± from intake structure to lower gate house, and 70 feet ± from lower gate house to discharge at stream.

Closure: One manually operated valve at the intake structure and another at the lower gate house.

Access: Same as 1.3h above.

Regulating facilities: Mud pipe valve at intake open at all times, whereas valve on the mud pipe at lower gate house is kept closed except in emergency.

SECTION 2

ENGINEERING DATA

2.1 Design

In 1917, John Henry, a junior engineer for the State of New York Conservation Commission (NYSCC), did some hydraulic computations for the design of the spillway overtopping section and spillway channel. He also performed an analysis of the stability of the gravity section of the dam. His computations along with his memorandum summarizing his findings are presented in Appendix E. However, on the first page of his memo he refers to 7 items that could have been very useful for further understanding of the design of this dam as envisioned in 1917, but unfortunately only Item #7 is available now.

Item #7 refers to 7 sheets of blue print drawings showing plans, sections and details descriptive of the dam, spillway channel, reservoir and structures appurtenant thereto. All these drawings are signed by W. E. Natanson, City Engineer, Rochester, N.Y. They were approved on August 3, 1917 by Division Engineer A. H. Perkins and Commissioner George D. Pratt of NYSCC, Albany, N.Y. A description of each of these drawings is given below.

Sheet #1 (Plate III) presents a general location map on a scale of 1" = 1 mile and plan of the reservoir on a scale of 1" = 100 feet. The number 73 acres marked on this drawing is apparently the size of the property, and should not be mistaken for the reservoir area, which has been checked and found to be close to 41 acres as reported in the Oct. 7, 1963 application, of the City of Johnstown, for reconstruction of the dam.

Sheet #2 (Plate IV) shows a plan of the earthen embankment, spillway overfall section, spillway channel and other appurtenant structures on a contour map with a contour interval of 2 feet. The following items shown on this drawing were probably eliminated from the final design and never constructed because they were not encountered during the field inspection:

i. 24" valve on the crest of the earthen embankment

ii. 16" valve and 16" C.I. bypass pipe on the right side of the spillway right abutment

iii. Footbridge at the end of the spillway channel

iv. Expanded section of the spillway channel downstream of the footbridge

v. Aerator and items associated with it.

Sheet #3 (Plate V) provides a longi'udinal section through the embankment and the concrete spillway. It displays the original ground surface, the approximate bottom of the concrete core wall, and wooden sheet piling under the concrete spillway to approximately elevation 1033 feet. This drawing also shows four typical cross sections of the embankment at Stations 0-200, 0-60, 0+86 and 2+05. The 24-inch valve and the related vertical concrete shaft from the crest of the embankment, shown in cross section at Station 2+05, have not been constructed.

Sheet #4 (Plate VI) shows plan and typical sections of the spillway and the spillway channel.

Sheets #5 through #7, not reproduced here, show features not directly related to the safety of the dam, such as plan and cross section of the aerator (not constructed), longitudinal section of the spillway channel, cross sections of the spillway channel at different stations, gate house, etc.

Two drawings (the first reproduced as Plate II) dated September 1963 and prepared by Morrell Vrooman Engineers, Gloversville, N.Y., and the October 7, 1963 application by the City of Johnstown, are the only source of information about raising the crest of the spillway from elevation 1054 feet to elevation 1055.25 feet.

There are only minimal hydrological computations and there are no design parameters available for checking the stability of the earth embankment.

2.2 Construction

There are no formal records of original construction or remodelling of the spillway crest available. Concrete strength tests, reported in 1919 (3000 psi) are included in Appendix E.

2.3 Operation

The water level in the reservoir is recorded by Mr. Tim Newhouse twice a day for seven days a week. Downstream flow is monitored for water supply purposes.

As a general operational procedure, water is

drawn down to approximately 1 foot below spillway crest in late fall and maintained at that level through the winter. By April, the pool is purposely drawn down to about 6 feet below spillway crest for safely containing the spring and summer floods and passing only a slight amount. In autumn, the pool level is drawn down again. The highest water level reported is 2 feet over top of spillway in the early 1970s. Usually water does not flow over the spillway.

A list of "Instructions for State of Emergency Conditions" is hung on the office wall of the Department of Water in the City of Johnstown and is included in Appendix E.

2.4 Evaluation

a. Availability

Engineering data were provided by the New York State Department of Environmental Conservation (NYSDEC) and by the owner, the City of Johnstown. The owner's City Engineer, Mr. Charles Ackerbauer, and the valve crew explained and demonstrated operational procedures during the visual inspection.

Mr. John Henry, a junior engineer, in his letter of July 24, 1917 to Mr. A. H. Perkins, Division Engineer, acknowledges the receipt of 7 items concerning the design of this dam. Items 1, 2 and 3, namely the original application for construction, the engineer's report, and specifications, could supply additional information on the original concept of the design. However, these documents were not available for review.

b. Adequacy

Although computations with respect to stability analysis of the gravity section of the dam are available, there is no justification provided for the basic assumptions made. For example, stability of the dam is dependent on the uplift pressures and the properties of the foundation material, yet neither of them has been investigated or evaluated. There is no information on the embankment material and its properties. Similarly, the nature and amount of hydrology data is also very limited. Consequently, the stability of the earth embankment could not be analyzed, and the overall assessment is primarily based on the following factors:

i. Visual observations made on the day of the inspection

ii. Overall assessment of the available data

iii. The analyses performed using hydrologic modelling data available in Upper Hudson and Mohawk River Basins Hydrologic Flood Routing Models, prepared for the Department of Army, New York District Corps of Engineers, by Resource Analysis, Inc. in 1976.

c. Validity

There are a number of items shown on the drawings which were not constructed as determined by visual inspection. This raises a question whether the concrete cutoff wall in the earth embankment was constructed or not, because excessive seepage was observed, on the downstream side, at the left abutment and along the toe of the embankment.

2.5 Geology

a. General Geology

The damsite and lake lie in southern Fulton County, New York. There is an extensive cover of glacial deposits at the surface. The underlying lithology is unknown.

There is a normal fault about a mile east of the dam, with the dam on the downthrown side. The fault trends north-south. There is a linement about 2 miles west of the dam.

The region underwent glaciation during the Wisconsin stage and is part of the glaciated Adirondacks.

b. Local Geology (Interpreted from stereo pair air photos)

Soil cover appears deep. The rock type beneath the glacial cover is unknown. The downstream channel looks dry and fairly free of vegetation.

The lake slopes look very steep. There are indications of siltation near the north inlet shore. In the photos, the lake level appears much lower than the dam spillway.

There is an apparent fault line about 2,500 feet downstream. The faultline is traceable about 4,000 feet in a southerly direction.

There were no geologic features (stratification, faults, cavities, etc.) detected or suspected that could be expected to adversely affect the dam or its appurtenant structures.

SECTION 3

VISUAL INSPECTION

3.1 Findings

a. General

The reservoir was inspected on July 20, 1978, a sunny warm day with 80°F temperature. The inspection team consisted of Messrs E. A. Nowatzki and G. S. Salzman from Converse Ward Davis Dixon, and Messrs C. Ackerbauer, T. Newhouse and R. Lake from the City of Johnstown.

The approach to the reservoir is through a dirt road which is in good condition; it runs in a northerly direction towards the reservoir from a light-duty Old State Road. The reservoir is locally called "High Daddy". This reservoir is one of the sources of water supply for the City of Johnstown. Cork Center Reservoir was the only component of this supply system inspected, and it appeared to be in generally good condition, except for the seepage noted on the downstream side of the earthen dam.

The overflow from the reservoir during floods passes over the concrete spillway, whereas normally the only outflow is through the 24-inch C.I. supply line, and in case of emergencies another 24-inch "mud pipe" can also be opened to discharge excess water.

b. Dam

The approach road leads to the left abutment of the earthen embankment from whence a 10-foot wide road (Fig. 1, Appendix D) in fairly good condition continues over the crest to the junction of the right abutment with the spillway. Just before the spillway, the embankment takes a 90° turn such that the direction of the spillway is perpendicular to that of the embankment. Over the spillway, there is a 3-foot wide concrete footbridge (Figs. 2 and 3, Appendix D) leading to a small length of embankment, which is heavily wooded and appears to be natural ground.

Near the center of the main embankment, there is an angle-iron access bridge leading to a concrete intake structure (Fig. 1, Appendix D). The access bridge was constructed in 1963, and is generally in good condition but rusted. It needs scraping and painting, but one should be careful to avoid contamination of water with lead compounds or other potentially detrimental material.

The rip-rap (Fig. 4, Appendix D) on the upstream face comes up to approximately spillway crest level, and its visible portion is in good condition. There is vegetation above the rip-rap to the edge of the crest, and small woody growth is developing which should be checked at this stage (Figs. 1 and 4, Appendix D). The downstream face of the embankment is very heavily wooded with tall pines and deciduous trees (Fig. 1, Appendix D). Widespread animal burrows were also observed on the downstream slope. Animal burrows and decaying roots of any dying trees could create seepage channels through the embankment, but development of such a condition is inhibited by the presence of the concrete cutoff wall.

Inspection of the downstream face of the embankment at the left abutment (junction with natural soil) revealed a 10' x 10' wet zone about half way down. There was no flow noticed. Farther down to the left, water disappears and emerges again several times along the junction of the embankment with the natural ground surface (the crotch). The presence of several small springs indicates that the flow is coming from the embankment. Seepage also extends along the toe of the dam to the right abutment but does not go up the right abutment crotch. The bottom 5 to 10 feet of slope is wet and spongy. Water can be heard running below the ground surface before emergence. The springs (Fig. 5, Appendix D) empty into the creek where the 24-inch supply and mud pipes empty. Seepage water is clear, with no sloughing or erosion observed. Decay of vegetation in the seepage path (Fig. 6, Appendix D) indicates it has been going on for some time. The observation of seepage indicates that the cutoff wall is not fully effective.

c. <u>Appurtenant Structures</u>

1. Gravity spillway section and spillway channel: Inspection of the concrete gravity overflow section and the spillway channel led to the following observations:

i. The overflow section itself (Fig. 2, Appendix D) appears to be in generally good condition with a moderate amount of spalling and erosion of its downstream slope and piers (Fig. 7, Appendix D).

ii. About one-third of the way down to the base of the overflow section along its junction with the left wing wall, there are signs of minor seepage indicated by wetness (but not flow) (Figs. 8 and 9, Appendix D). On top of the left wing wall of the spillway, there are signs of pressure grouting which might have been done to

control leaks, or the concrete may be the result of surface guniting; there is a scratched date of 1963.

iii. Minor spalling and scaling of wing walls has started (Fig. 8, Appendix D).

iv. The footbridge over the spillway is scaling, with a large spall near the midspan exposing steel (Figs. 3 and 7, Appendix D).

v. The spillway discharge channel (Fig. 10, Appendix D) has minor spalls at wall joints and on the wall. At the lower end of the spillway, there is a major spall on the left wall.

vi. The last floor slab of the spillway channel is cracked and seepage comes out from below the slab at its lower lip (Fig. 11, Appendix D).

2. Intake structure and outlet pipes: The intake structure consists of a vertical concrete shell with an 11-foot inside diameter and 6-inch thick wall. The top of the intake structure has a concrete floor with a central rectangular entrance to the intake box (Fig. 12, Appendix D). The entrance is covered with steel plate and locked with a chain. There is a circular pipe railing around the platform and three valve stems for the gates on top of the platform (Fig. 13, Appendix D). Gates for the supply line are located at 17-foot and 25-foot depths below the intake platform. The mud pipe extends 30 feet farther from the intake into the lake and is at a 40-foot depth. All three gate valves were turned and found to function smoothly. The intake box cover was removed, but the valves could not be seen because water inside the intake structure was at the pond level. The gate platform and intake structure wall are moderately spalled and scaled (Fig. 13, Appendix D).

The supply and mud pipes traverse under the embankment, through the lower gate house, and then empty into the natural stream (Fig. 14, Appendix D). The steel on the pipes at the outfall looked good.

3. Gate house: The gate house appeared to be in good condition. It has a wooden floor with stairs leading to lower level for access to gates. No lighting arrangement exists. There are four valves in the gate house. Two 24-inch valves are on the main water supply line, one 24-inch valve on the mud line, and one 16-inch valve on the pipe connecting the two lines. All valves

were cracked and found to be functioning smoothly. They are well maintained. Very muddy effluent resulted at the cracking of the mud pipe valve which was closed immediately. The lower gate house floor is a little wet because of seepage into the house, but the pipes and valves under the floor look good and dry.

d. Foundation

The foundation for the dam was not observed. Our geologic evaluation of the site indicates that there is an extensive cover of glacial deposits at the surface. (See Article 2.5)

e. Reservoir Area

The reservoir area is heavily wooded except for one naturally sandy area that is covered with moss and scrub only (Fig. 15, Appendix D). Side slopes of the reservoir are steep; about 1½ horizontal to 1 vertical. However, there is no evidence of slope failure. There is a moderate amount of sedimentation at the upstream entrance and an indication of sedimentation at the dam, suggested by the turbidity of water released from the mud pipe.

f. Downstream Channel

The downstream channel (Fig. 16, Appendix D) is clear of any obstruction or debris. The upper reaches of the channel form part of the water supply channel to the chlorination house. The valley and slopes are generally wooded and appear stable.

3.2 Evaluation

Seepage was observed starting about half-way down the downstream slope of the embankment along its left abutment, continuing along the toe to the right abutment, but does not go up along the crotch of the right abutment. The amount of seepage appears to be substantial, indicating that the cutoff wall is not entirely effective. Although there are indications that this condition has existed for some time and there are no signs of erosion or failure, further investigation is definitely advisable.

About one-third of the way down to base of the spillway overflow section along the junction with the left wing wall, there are signs of minor seepage indicated by wetness and discoloration of the concrete. At present this situation is not of major concern, but it could worsen

with time and the structural concrete may be affected by frost action.

The presence of large trees on the embankment slopes of earthfill dams ordinarily poses a potentially dangerous condition.

a) If the trees are shallow rooted, they could blow over in a major storm, carrying part of the embankment with them.

b) If the trees are deep rooted, the root systems may extend transversely through the embankment. Death of the trees and subsequent decay of the root systems may result in the formation of water passages (pipes). Such pipes provide natural channels for the seepage of water through the embankment; this may result in erosion of the embankment or in the generation of seepage forces that would adversely affect the stability of the slope.

c) The trees on the downstream face of the subject dam appeared to be well established. A study should be made to establish whether the trees are shallow rooted or deep rooted. If they are shallow rooted, removal is in order. If they are deep rooted, removal would be potentially more dangerous than leaving them in place; for this dam, the danger is substantially mitigated by the presence of the concrete cutoff wall.

SECTION 4

OPERATIONAL PROCEDURES

4.1 Procedures

Personnel of the City of Johnstown Water Department (JWD) informed us that the water level in the reservoir is recorded on a twice-a-day basis for 7 days a week. Mr. Tim Newhouse of the Water Department is assigned to this study, in addition to recording water levels at other locations such as at the weir, farther downstream. There are no written procedures made available; however, we were informed by JWD personnel that, according to their established practice, water is lowered in late fall to about 1 foot below the spillway crest and retained there throughout the winter. The lake freezes in winter and the normal ice thickness is between 18 and 24 inches. By April, the pool level is further lowered to about 6 feet below spillway crest to receive and safely pass the spring and summer floods. Normally, water does not flow over the spillway; however, the highest water level reported in the early 1970s is about 2 feet above the spillway crest.

Two valves for the 24-inch supply line and one for the mud pipe, all three located at the intake structure, are kept open at all times. They are worked once a year to verify the satisfactory functioning of the mechanism. Water outflow from the reservoir is regulated by one of two supply line 24-inch valves located in the lower gate house. The other 24-inch valve, also located in the lower gate house, but operating the mud pipe, is kept closed except in emergencies. Water from the supply line (and mud pipe) drains out into a natural channel which leads to the downstream diversion works.

4.2 Maintenance of Earth Embankment

The only apparent maintenance is periodic cutting of vegetation on the upstream face. Except for downstream seepage along the left abutment and toe, the earth embankment seems to be in generally good condition. There are no visible signs of sloughing, erosion or cracking of the embankment. The rip-rap on the upstream face extends up to the spillway crest level with no visible failures. There are extensive animal burrows but they should not pose any hazard in view of the barrier provided by the concrete cutoff wall. The downstream edge of the crest, and the downstream face are heavily wooded with

tall pines. There are some deciduous trees on downstream slope. Tree growth is starting on upstream face above rip-rap line, where cutting was apparently omitted for a while. The roadway on top of the dam is in good condition.

4.3 Maintenance of Concrete Gravity Structure and Spillway Channel

Structurally, the gravity structure appears to be in good condition but needs general maintenance. The last general maintenance was apparently in 1963. The downstream face and piers have spalled and eroded moderately. Minor spalling and scaling of wing walls have started. The bridge is badly scaled with a large spall near midspan, exposing steel. About one-third of the way down to the base of the spillway overflow section along the junction with the left wing wall, there are signs of minor seepage indicated by wetness and discoloration. The spillway channel has minor spalls at wall joints and on the wall. At the lower end of the spillway channel there is a major spall on the left wall. The last floor slab of the spillway channel is cracked, and seepage comes out from below the slab at the lower lip.

4.4 Maintenance of Operating Facilities

The access bridge to the intake structure is rusted and does not appear to have been painted since installation in 1963. The gate platform at the intake structure is moderately spalled and scaled. Valve stems at the intake structure and gate house turned, and were observed to function very well with little or no slack. At the outfall, the metal pipes appear to be in good condition. The lower gate house floor is a little wet from seepage into the house, but pipes and valves under the floor look good and dry. There is no lighting arrangement at the lower gate house.

4.5 Warning Systems in Effect

The general condition of the dam and its appurtenant structures are checked daily as part of the pool elevation monitoring procedure. "Instructions for State of Emergency Conditions" have been written and posted on the office wall of the Department of Water so that all employees are familiar with them. Names of people to be notified in case of emergency also appear on those instructions. Access to the controls is maintained in the winter by plowing away the snow from the access road at high priority.

4.6 Evaluation

Functionally, all parts seem to be in good working condition. Maintenance of the operating facilities appears to be satisfactory. However, some routine maintenance needs attention. Small woody growth on the upstream slope above rip-rap level should be taken out and no more woody growth allowed in the future. Minor concrete repairs on the spillway, wing walls, footbridge, spillway channel and intake structure should be undertaken. The rust on the access bridge to intake structure needs scraping followed by painting (being careful with the use of lead components or other potentially deleterious materials).

As a part of general maintenance, the mud pipe should be flushed at least annually to avoid clogging of its upstream open end.

Seepage between the spillway and the left abutment should be eliminated by injection grouting.

Provision of an electrical connection for light at the gate house or any other source of light such as a flashlight is very essential.

Instructions for handling emergency situations appear to be adequate, except for exceptionally high flows, as discussed later.

SECTION 5

HYDRAULICS AND HYDROLOGY

5.1 Evaluation of Hydraulic Features

a. Design Data

The dimensions of the overflow spillway and the spillway channel are found on, or can be scaled from, Plates II and VI. Based on some referenced tables, that we are not familiar with, the overflow spillway capacity in the original file was calculated as 1630 cfs (Appendix E) for a 4-foot head, which is not too far from our calculation of the overflow spillway capacity as 1814 cfs (Appendix C) for the same head. However, with the raised crest level, the available head over the spillway is 3.75 feet without overtopping the earthen embankment, and for that head the spillway capacity according to our computations drops to 1650 cfs (Appendix C), which is very close to the original design.

Using Chezy and Kutter formulae, the original file arrives at 1670 cfs (Appendix E) as the capacity of the spillway channel for the designed dimensions and slope and water depth of 5 feet. We concur with this result.

The two 24-inch pipes have been shown to be capable of discharging a total of 150 cfs for a head of 20 feet of water. Their method and computations are correct but the available head of water at the upstream end of the mud pipe is 40 feet. We have not been able to establish the depth of the supply line, but the upstream head may be assumed as a minimum of 25 feet. Our estimate of discharge through these two pipes is 320 cfs.

The original flow computations are presented in Appendix E and computations performed as part of this study are found in Appendix C.

b. Experience Data

A record of water levels at the reservoir is available, but no measurements of flow are available. The maximum observed head of water over the spillway crest was reported to be about 2 feet in the early 1970s.

c. Visual Observations

The pool elevation on the day of the inspection was 16 inches below that of the spillway crest, so the spillway was not observed in operation; however, there is no reason to believe that it would not function satisfactorily. The maximum height of water that the spillway can accommodate without overtopping of the dam is 3.75 feet. The gate valves on the supply and mud lines at intake and lower gate house were turned and found to function satisfactorily. Only a small amount of flow was being maintained through the supply line. Opening of the mud pipe gate valve was followed by very muddy effluent, so the valve was closed immediately.

5.2 Evaluation of Hydrologic Features

a. Design Data

For calculation of the flood discharge for this dam, in the original computations of 1917, an empirical formula developed by Mr. McKim, the then inspector of dams, has been used. This formula is based on the drainage area of the watershed and gives a flood discharge of 1314 cfs for this watershed. A more detailed analysis was attempted later but left incomplete. These computations are found in Appendix E.

To our knowledge, there are no gaging stations in the local basin. According to the Recommended Guidelines for Safety Inspection of Dams, Department of the Army, OCE, the recommended Spillway Design Flood (SDF) for the subject dam is the Probable Maximum Flood (PMF) since the dam is of intermediate size and poses a high hazard.

b. Experience Data

Information on the PMF for the Cork Center Storage Reservoir and its watershed was obtained from the Upper Hudson and Mohawk River Basins Hydrologic Flood Routing Models prepared in 1976 for the New York District of the U.S. Army Corps of Engineers (USACE) by Resource Analysis, Inc. In this study, the rainfall-runoff mathematical model HEC-1 was used to reconstitute the major historical floods and to simulate the Standard Project Flood (SPF). In addition to the SPF simulation, the rainfall pattern for Tropical Storm Agnes was transposed to fall directly on the basins under study, and the discharges resulting from this rainfall were determined by an application of the calibrated model. In a telephone conversation with Mr. Thomas Smyth, USACE New York District, we

were informed that for Phase I hydrologic analyses, the Probable Maximum Flood (PMF) could be considered as twice the SPF.

The Cork Center Storage Reservoir and its drainage basin were located within Subarea 22 of the Mohawk Basin, Little Falls, N.Y. to Mouth. Computations for routing the PMF through the Cork Center Storage Reservoir are found in Appendix C of this report.

c. Visual Observations

The maximum observed flood over the spillway crest is about 2 feet in the early 1970s. Normally, the water does not flow over the spillway. This appears to be verified by the observable water marks on the wing walls at about spillway crest level (Fig. 2, Appendix D). We were informed by JWD personnel that the pool of the Cork Center Storage Reservoir is lowered to about 6 feet below the crest level. This may explain why heavy rains of the early 1970s were passed without difficulty.

d. Overtopping Potential

The computations in Appendix C indicate that the subject dam will be overtopped by the PMF. The maximum height of water that can flow over the spillway without the dam being overtopped is 3.75 feet. At that height the spillway passes approximately 1650 cfs. The routed PMF is approximately 3630 cfs. Therefore, the spillway can pass only 45 percent of the PMF.

e. Spillway Adequacy

The results of the hydrological analysis indicate that the spillway capacity is inadequate with respect to passing the PMF, and the topping of an earth dam often results in the rapid washout of a dam section. In addition, the spillway is considered seriously inadequate because it satisfies all of the following conditions set forth in DAEN-CWE-HY Engineer Technical Letter No. 1110-2-234 dated 10 May 1978:

1. There is high hazard to loss of life from large flows downstream of the dam.

2. Dam failure resulting from overtopping would significantly increase the hazard to loss of life downstream from the dam from that which would exist just before overtopping failure.

3. The spillway is not capable of passing one-half of the Probable Maximum Flood without overtopping the dam and potentially causing failure. It may, however, be pointed out that if the supply and mud pipes are kept open, the total discharging capacity will increase to approximately 54% of the Probable Maximum Flood.

SECTION 6

STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

a. Visual Observations

Visual observations of the earth embankment disclosed considerable amount of seepage along the downstream toe. Seepage starts at the junction of the left embankment with the virgin hillside about half-way down the downstream slope. It turns into flowing water along the toe and extends to the right abutment but does not go up along the crotch of the right abutment. The decay of vegetation in the seepage path indicates it has been going on for some time. The bottom 5 to 10 feet of the slope is wet and spongy but there is no sign of sloughing or erosion. The vertical and horizontal alignments of the embankment appear to have been maintained, and there is no evidence of cracks. Some roots of the pines growing on the downstream slope cross the embankment crest transversely; this condition may not exist at depth because of the presence of the concrete cutoff wall.

About one-third of the way down to base of the overflow spillway and along the junction with the left wing wall, there are signs of minor seepage indicated by wetness. At present, this situation is not of any concern, but it could worsen with time and the structural concrete could deteriorate due to frost action.

b. Design and Construction Data

The original computations of 1917, presented in Appendix E, were reviewed. Generally, these computations were found appropriate, except that in one case, the coefficient of active earth pressure was used for calculating passive resistance. Assuming 66% uplift pressure on the base of the dam, it was found safe against overturning about the toe. However, it was found unsafe with respect to sliding because the passive resistance to the cutoff walls was not considered. At a later stage in those computations, resistance to cutoff wall was taken into account but with an active earth pressure coefficient instead of a passive earth pressure coefficient, and the resistance of the downstream key was still not accounted for. Under these conditions the spillway was found to be just safe against sliding with a Factor of Safety of 1, assuming a coefficient of friction of 0.45 between the

concrete and the foundation soil. In the original analysis, ice pressure was neglected.

In the present analysis, the stability of the spillway section prior to 1963 and after remodelling in 1963 has been analyzed for various conditions of pool elevations and ice thrust. The results are shown in a tabular form in Appendix C. The uplift pressure on the base of the dam was taken as 50%, and the coefficient of friction against sliding as 0.3. The only critical condition encountered is in overturning when the pool is at spillway crest level and ice thrust of 4 kips/linear foot is applied 1 foot below the crest. It is, therefore, advisable that pool level during winter be kept at least 3 feet below the spillway crest level. This is mitigated somewhat by the fact that the spillway section has received ice thrusts for about 15 years without visible harm.

Since no information was available regarding the nature of the embankment and foundation materials and their properties, neither stability nor seepage analyses for the embankment could be performed as part of this study.

The present operational procedure provides for the water supply pipe and the mud pipe to be under pressure beneath the dam. This is considered undesirable, as a leak in the line could eventually result in washout of embankment soil, which could endanger the integrity of the dam.

c. Operating Records

None available.

d. Post Construction Changes

In 1963, the crest of the overflow spillway section was raised by 15 inches according to the drawing on Plate II. At the same time, an angle iron steel access bridge was constructed from the embankment crest to the intake structure. Probable pressure grouting or surface guniting around the spillway is indicated about the same time by a scratched date of 1963.

e. Seismic Stability

The Cork Center Storage Reservoir is nominally located on the border between Seismic Zone 1 and Seismic Zone 2 according to the Algermissen Seismic Risk Map. The

USACE guidelines suggest that in the event of doubt about the proper zone, the higher zone should be used. Although earthquakes that cause moderate damage can be expected to occur in Zone 2, the design and construction practices conventionally used for small earth dams are considered to be adequate in areas of low seismicity and the safety factors used for static conditions should preclude major damage for all but the most catastrophic earthquakes. However, no computations were performed to evaluate the effect of earthquakes on the subject dam.

SECTION 7

ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 Dam Assessment

a. Safety

Visual inspection of the system and a review of the limited available engineering data indicate that the dam embankment and the overflow spillway are in generally good condition and functioning satisfactorily at this time. Our approximate hydrologic/hydraulic calculations indicate that the discharge capacity of the overflow spillway is seriously inadequate according to the OCE screening criteria. Although no signs of sloughing, erosion or cracking of the earthen embankment were observed, substantial amount of seepage along the downstream toe deserves further investigation. The stability of the embankment may then be analyzed in the light of the findings.

b. Adequacy of Information

The information available to us is not adequate for a detailed analysis of the stability of the embankment including seepage effects. However, the stability of the overflow spillway has been verified with reasonable degree of certainty, although the nature and properties of the foundation material are not clearly defined. Since there were no hydrologic data available, our assessment of the overtopping potential is based solely on transpositioning modelling results from nearby areas to the subject drainage basin.

c. Urgency

Inasmuch as the spillway capacity appears to be seriously inadequate according to the OCE screening criteria, there is some urgency in performing the additional study recommended below. Likewise, occurrence of seepage along the downstream toe of the embankment slope requires investigation at high priority. These investigations should be performed as soon as practicable; this should be within one year.

d. Necessity for Further Investigations

In view of the inadequacy of the overflow spillway in its inability to pass at least one half of

the computed PMF without overtopping the dam, and in view of the fact that overtopping in the case of earthfill dams is usually disastrous, the actual capacity of the spillway should be determined using more precise and sophisticated methods and procedures. This further investigation should be performed as soon as possible. Following this study, the need for and type of mitigating measures should be determined. Until such a study is completed, around-the-clock surveillance of the structure should be provided during periods of unusually heavy precipitation.

The occurrence of excessive amount of seepage along the toe of the embankment indicates that the cutoff wall has not been totally effective, which could be caused by any one or a combination of the following possibilities:

i. Seepage under the cutoff wall.

ii. Seepage through the cutoff that may have cracked.

iii. Seepage around the cutoff wall through virgin soil at the left abutment.

iv. Since there are no construction records available and also some items shown on drawings were not found during field inspection, the possibility that the cutoff wall was not constructed cannot be ruled out.

It is, therefore, recommended that borings be drilled through the downstream slope of the embankment, penetrating into the virgin soil, to establish the properties of both the embankment and the foundation materials. Later, piezometers may be installed in these borings to establish the seepage characteristics through the embankment. Subsequent stability analysis will provide a better understanding of the safety of this dam. If it were found safe, necessary protective measures to prevent piping failures (e.g. a subdrainage system and/or injection grouting) would then be recommended. Test pits should be dug along the centerline of the dam crest to verify the existence of the cutoff wall; the vertical dimension of the wall should be checked by coring.

7.2 Recommendations and Remedial Measures

a. Alterations/Repairs

1) Injection grouting for fixing leak between the downstream slope of the overflow spillway section and the left spillway wall should be undertaken.

2) All minor damages to concrete (spalling, scaling, etc.) at spillway piers, spillway downstream slope, abutment walls, spillway channel walls, concrete footbridge over spillway and intake structure, should be repaired.

3) Rust should be scraped from the intake structure access bridge and the bridge subsequently painted, taking care not to contaminate the water with lead or other deleterious materials.

4) The gate house shaft should be lighted, either by an electrical circuit or by a system of battery-operated emergency lights.

5) The low woody growth on the upstream face of the dam should be removed.

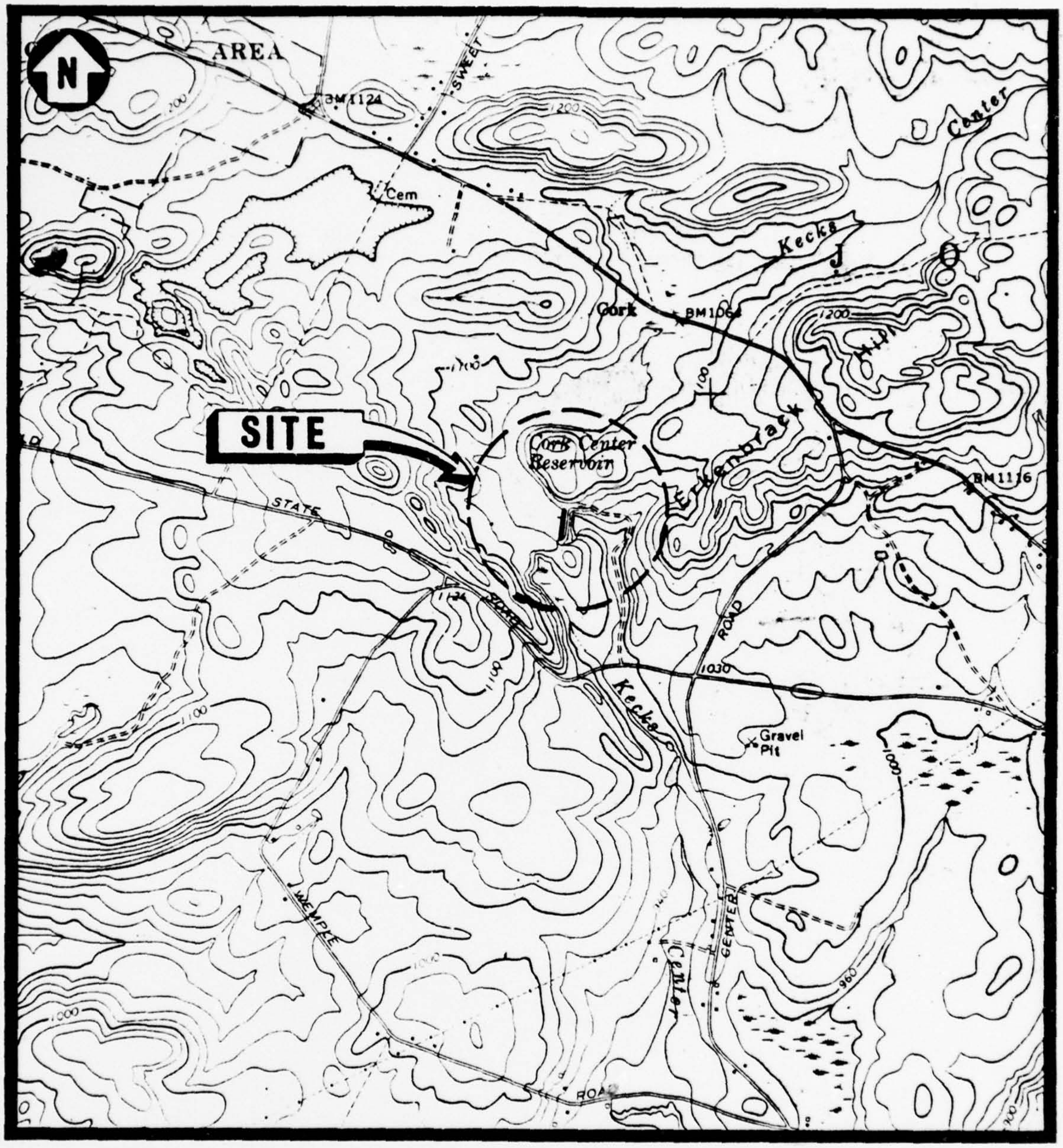
6) The large trees on the embankment should be investigated to determine whether they are shallow rooted or deep rooted. If shallow rooted, they should be cut down; if deep rooted, they should remain.

7) The flow from the lake should again be controlled from the intake structure rather than from the lower gate house, to avoid having pipes under pressure beneath the dam.

The remedial work recommended above is not critical in terms of urgency. It should be done as soon as practicable. Items 1, 5, 6 and 7 can be accomplished this year; all recommendations should be completed within the next three years.

b. Operations and Maintenance Programs

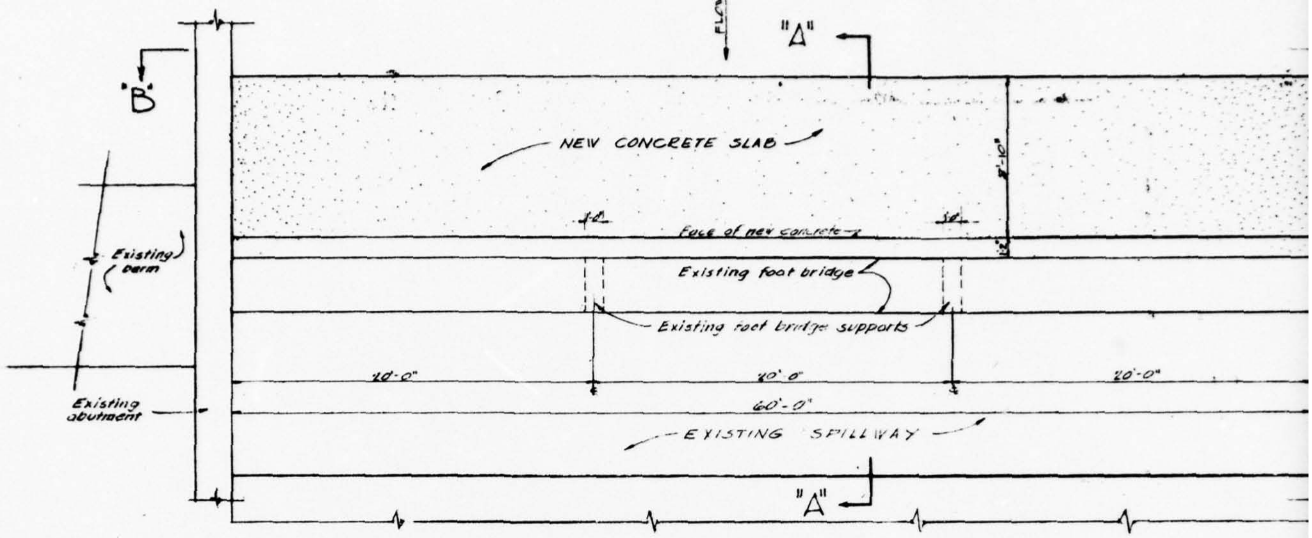
A specific program of periodic maintenance of the dam embankment and its appurtenant structures should be established and followed. This would include definite times for trimming of vegetation on the upstream slope, inspection and repair of concrete structures, testing of control valves for leakage, timely repair of the access road, etc. Periodically, water should be allowed to flow through the mud pipe (dirty water may be bypassed at the treatment plant) to avoid clogging of its open end with silt in the reservoir.



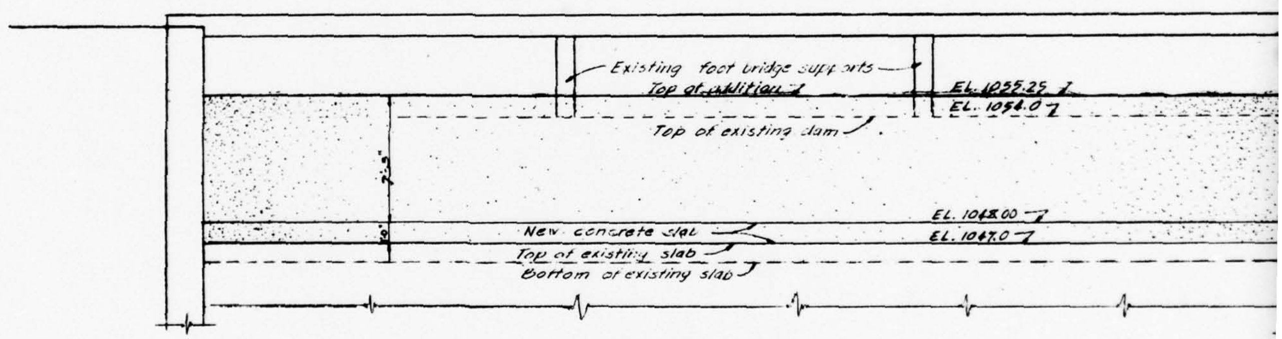
SCALE: 1"=2000'

MAP SOURCE: BASE MAP WAS ADAPTED FROM U.S. GEOLOGICAL SURVEY MAP, PECK LAKE, N.Y. QUADRANGLE, 7.5 MINUTE SERIES 1970. (BASE MAP MAY NOT REFLECT RECENT CARTOGRAPHIC CHANGES)

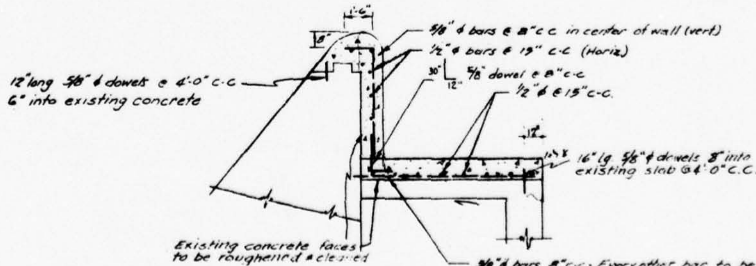
PLATE I SITE LOCATION MAP



PLAN
1/4" = 1'-0"



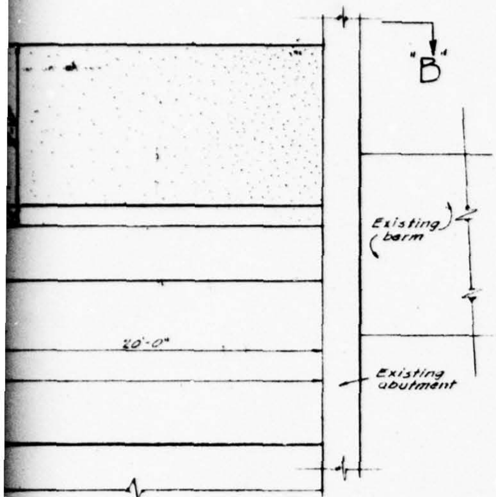
ELEVATION "B"-"B"
1/4" = 1'-0"

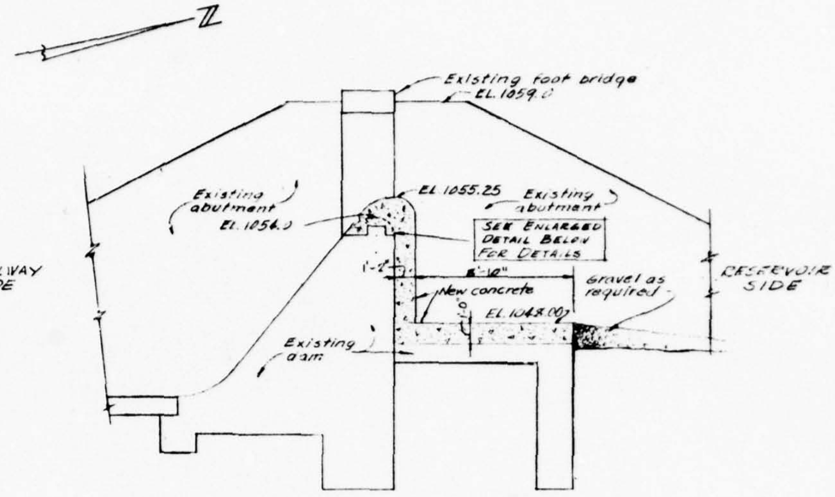


REINFORCING
1/4" = 1'-0"

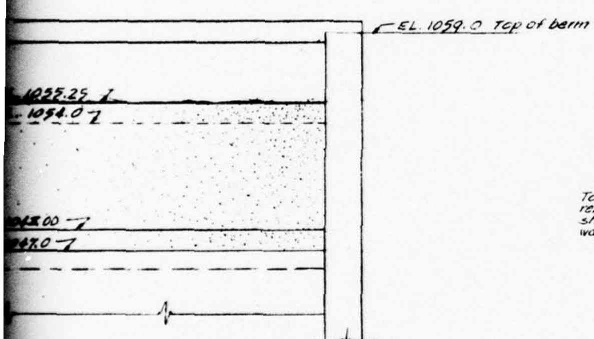
DRAWN BY	RWK-RHK
TRACED BY	RWK-RHK
CHECKED BY	E

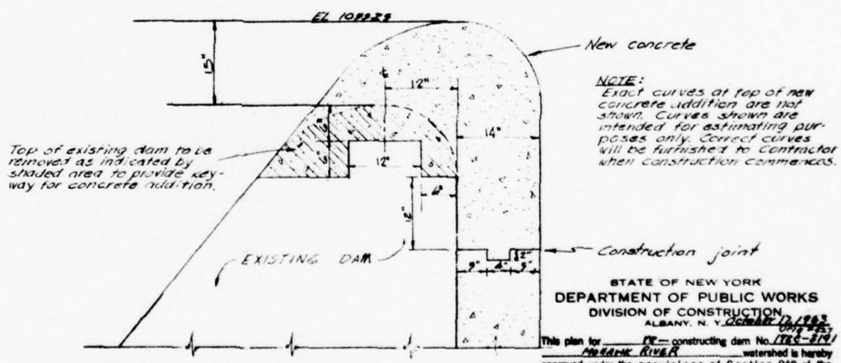
2





SECTION "A"-"A"
1/4" = 1'-0"





NOTE: Exact curves at top of new concrete addition are not shown. Curves shown are intended for estimating purposes only. Correct curves will be furnished to Contractor when construction commences.

ENLARGED DETAIL
1" = 1'-0"

STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION
ALBANY, N. Y. October 11, 1963

This plan for re-constructing dam No. 172C-3191 Mohawk River watershed is hereby approved under the provisions of Section 948 of the Conservation Law.

Examined and recommended to the Chief Engineer for approval.

ASSOCIATE CIVIL ENGINEER

APPROVED

CHIEF ENGINEER
Department of Public Works

By ______ Chief Engineer

GENERAL NOTES

1. Contractor shall be responsible for all dimensions and conditions at the construction site and be responsible for the accuracy of same.

CONVERSE WARD DAVIS DIXON
CONSULTING ENGINEERS
PLATE II AUGUST 1978

RECEIVED
DEP. CHIEF ENG.
OCT 14 1963
FLOOD CONTROL CANALS

CITY OF JOHNSTOWN
FULTON COUNTY, NEW YORK
WATER WORKS IMPROVEMENTS
RAISING CORK CENTER
STORAGE RESERVOIR DAM

SCALE: AS SHOWN SEPTEMBER 1963

Morrell Vrooman, Jr.
MORRELL VROOMAN ENGINEERS
GLOVERSVILLE, NEW YORK
LICENSE NO. N.Y. 22134

172C-3191 (Orig#427) MOHAWK RIVER

SHEET 1 OF 2 SHEETS

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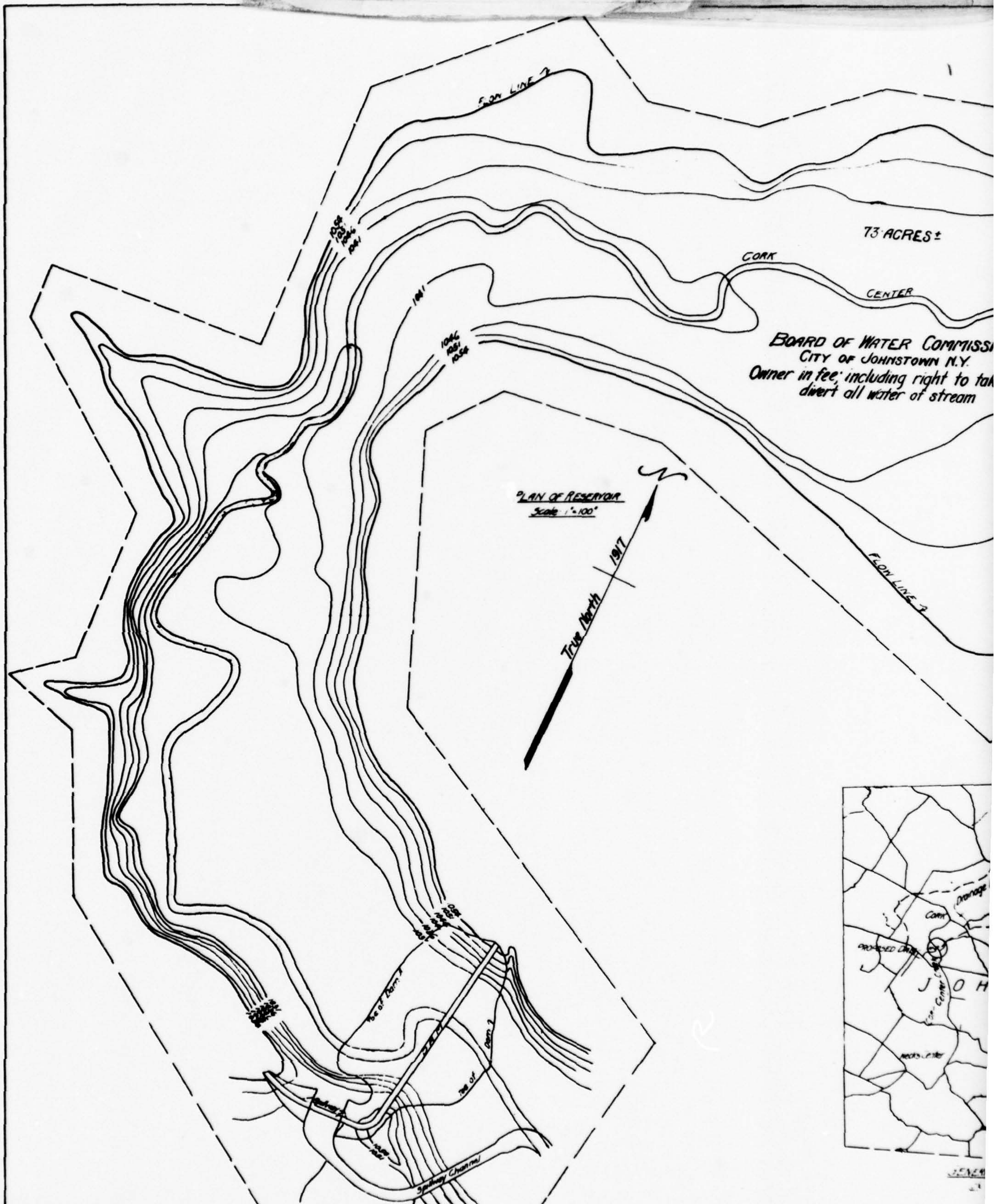

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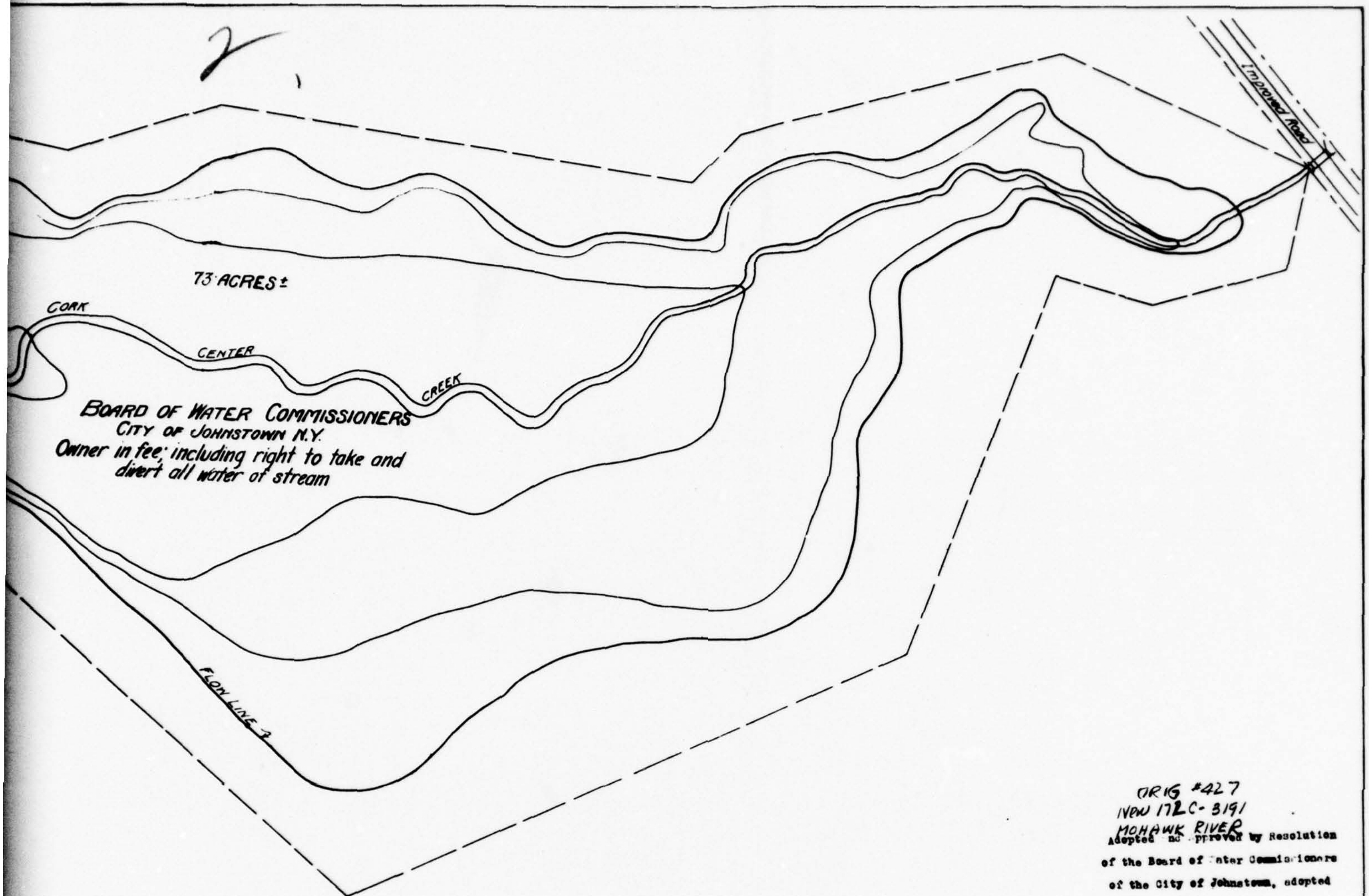
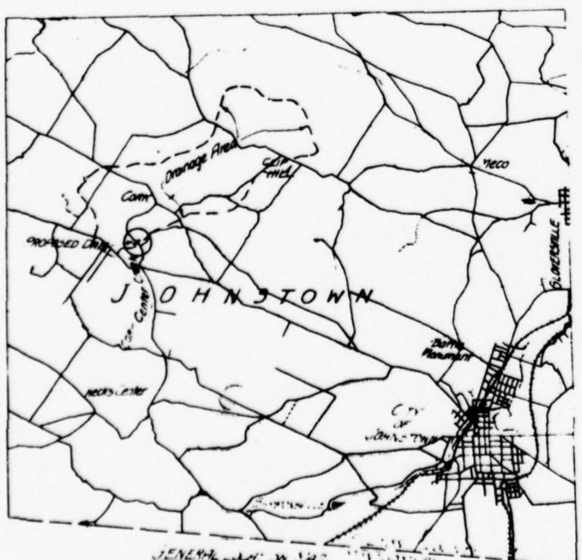


ORIG #427
NEW 172C-3191
MOHAWK RIVER

Adopted and approved by Resolution of the Board of Water Commissioners of the City of Johnstown, adopted July 9, 1917.

_____ President

_____ City Clerk.



GENERAL LOCATION MAP

JOHNSTOWN WATER SUPPLY

CORK CENTER CREEK

FULTON COUNTY, N.Y.

RESERVOIR AND LOCATION MAP

Scales As indicated

1

W.E. Malanson, City Engr. Johnstown, N.Y.

James P. Wells, Consulting Engr., Rochester, N.Y.

Made by

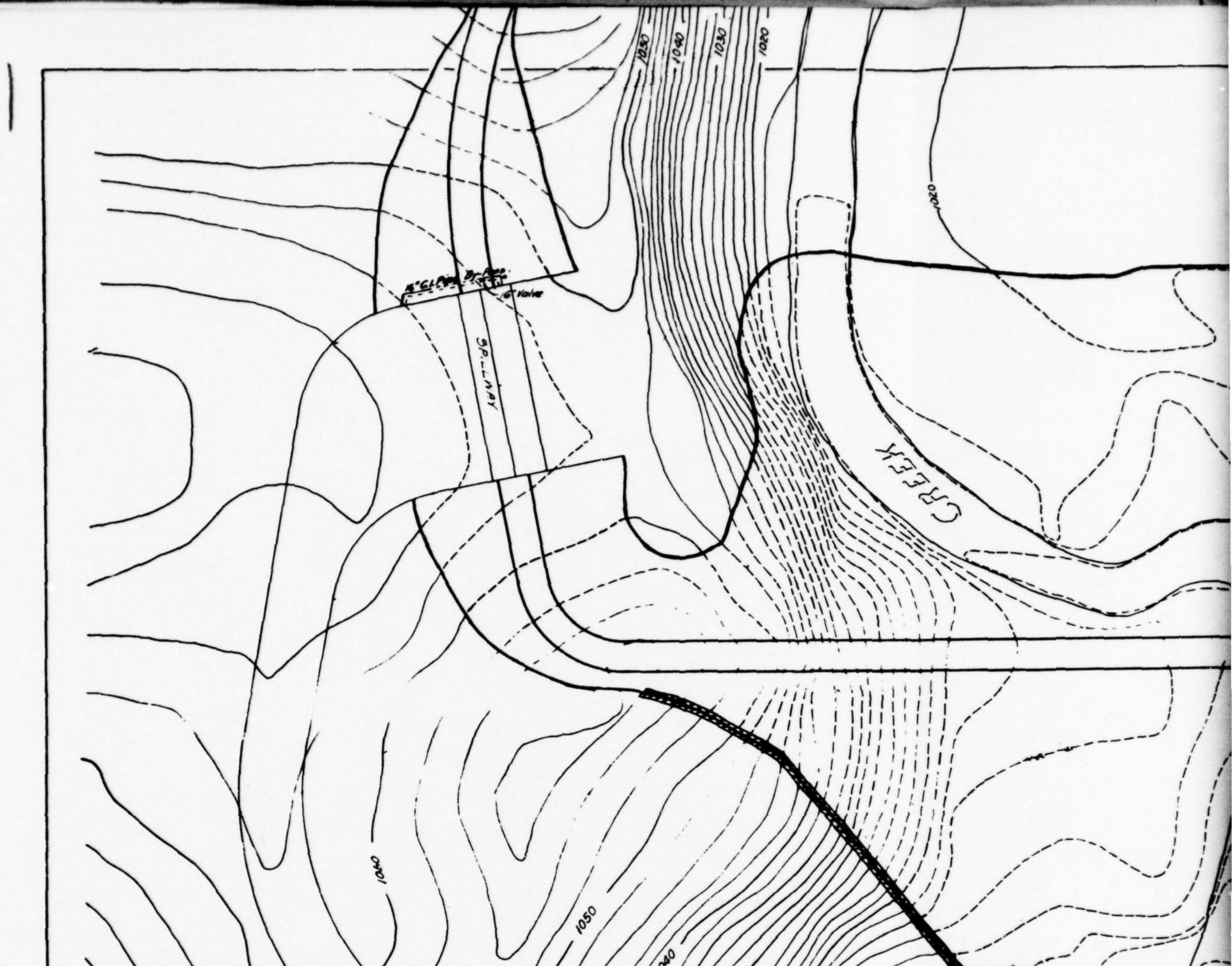
Traced by

Checked by

CONVERSE WARD DAVIS DIXON

CONSULTING ENGINEERS

PLATE III AUGUST 1978



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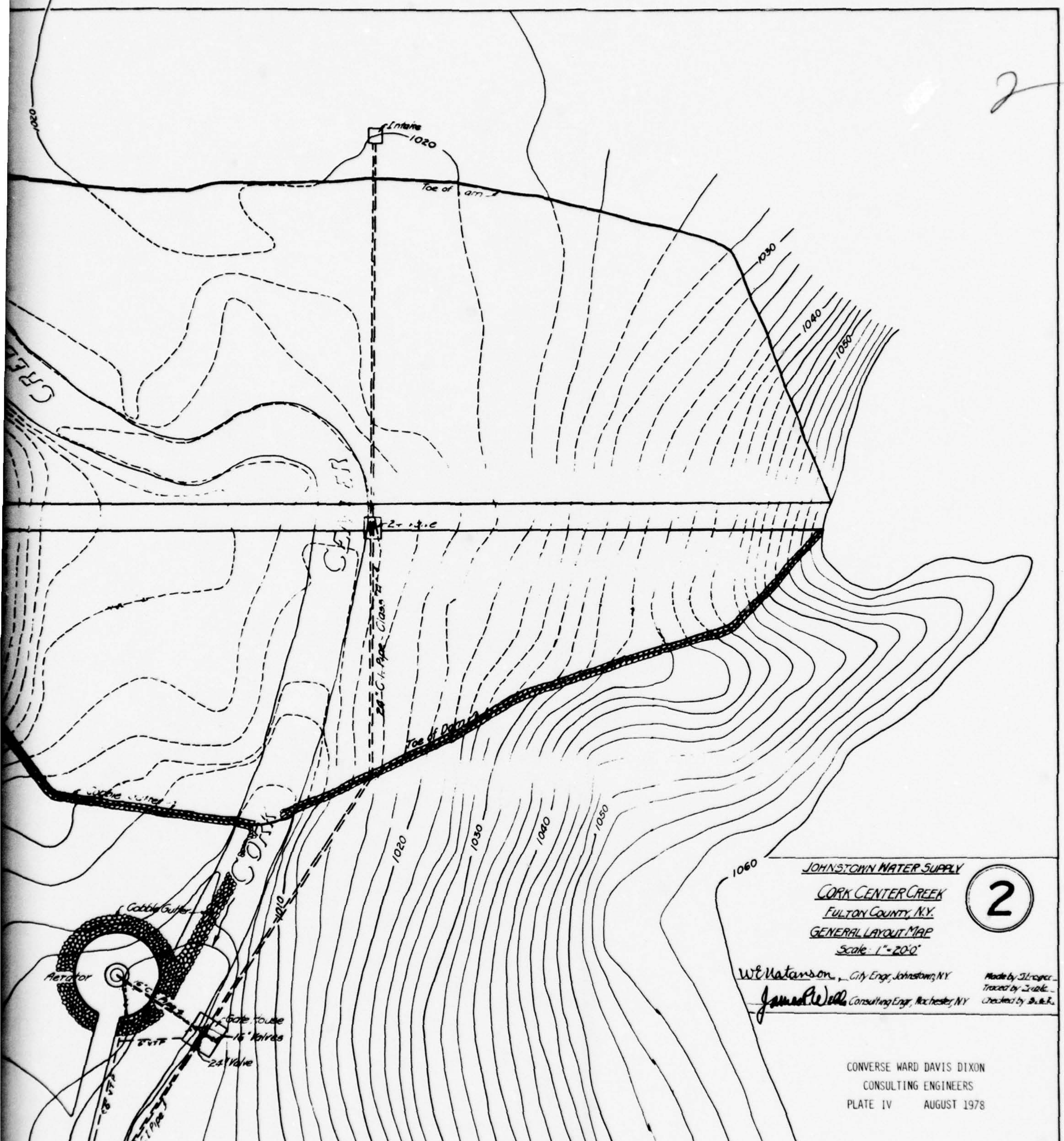

CONVERSE WARD DAVIS DIXON
CONSULTING ENGINEERS
PLATE IV AUGUST 1978

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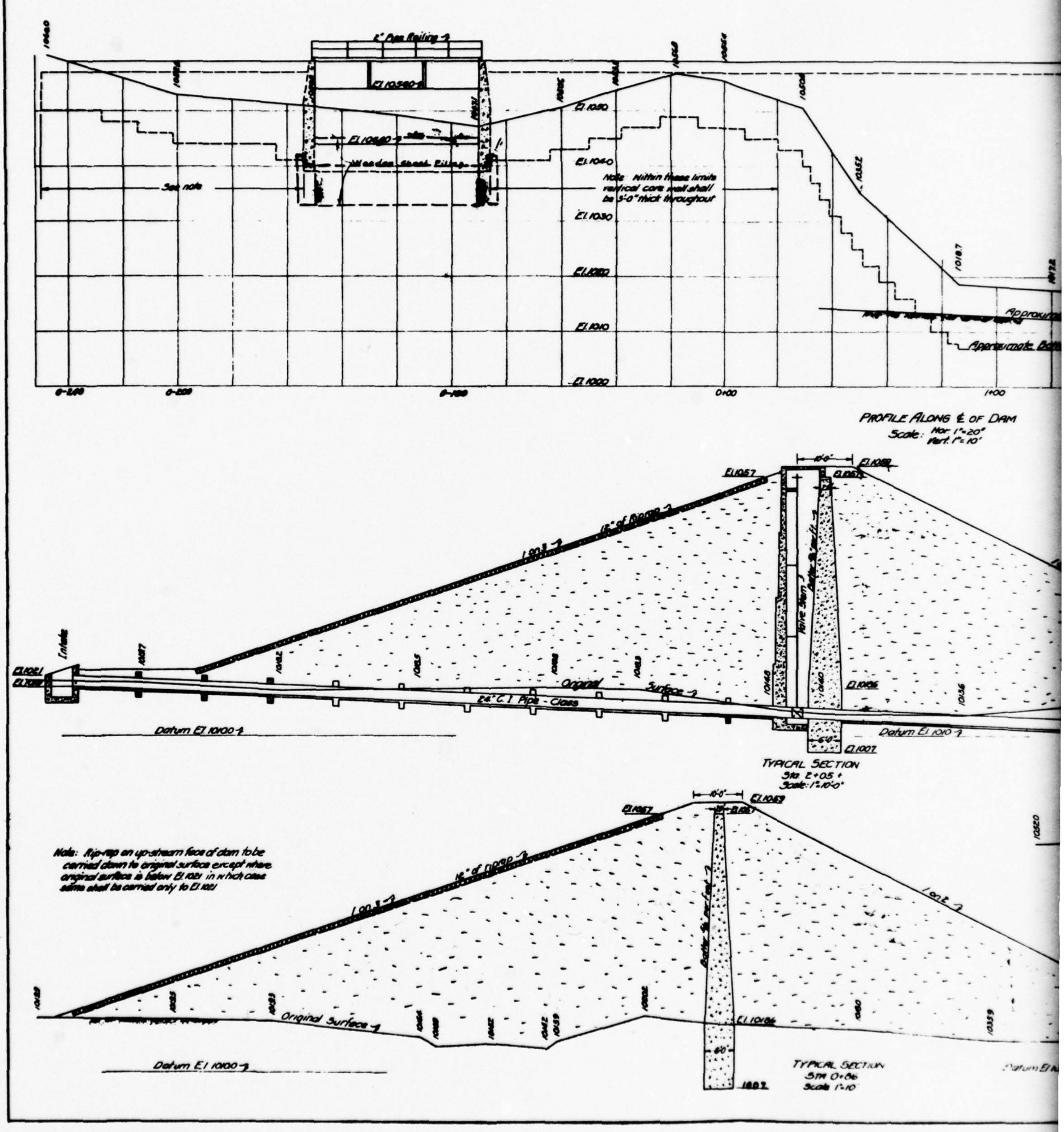

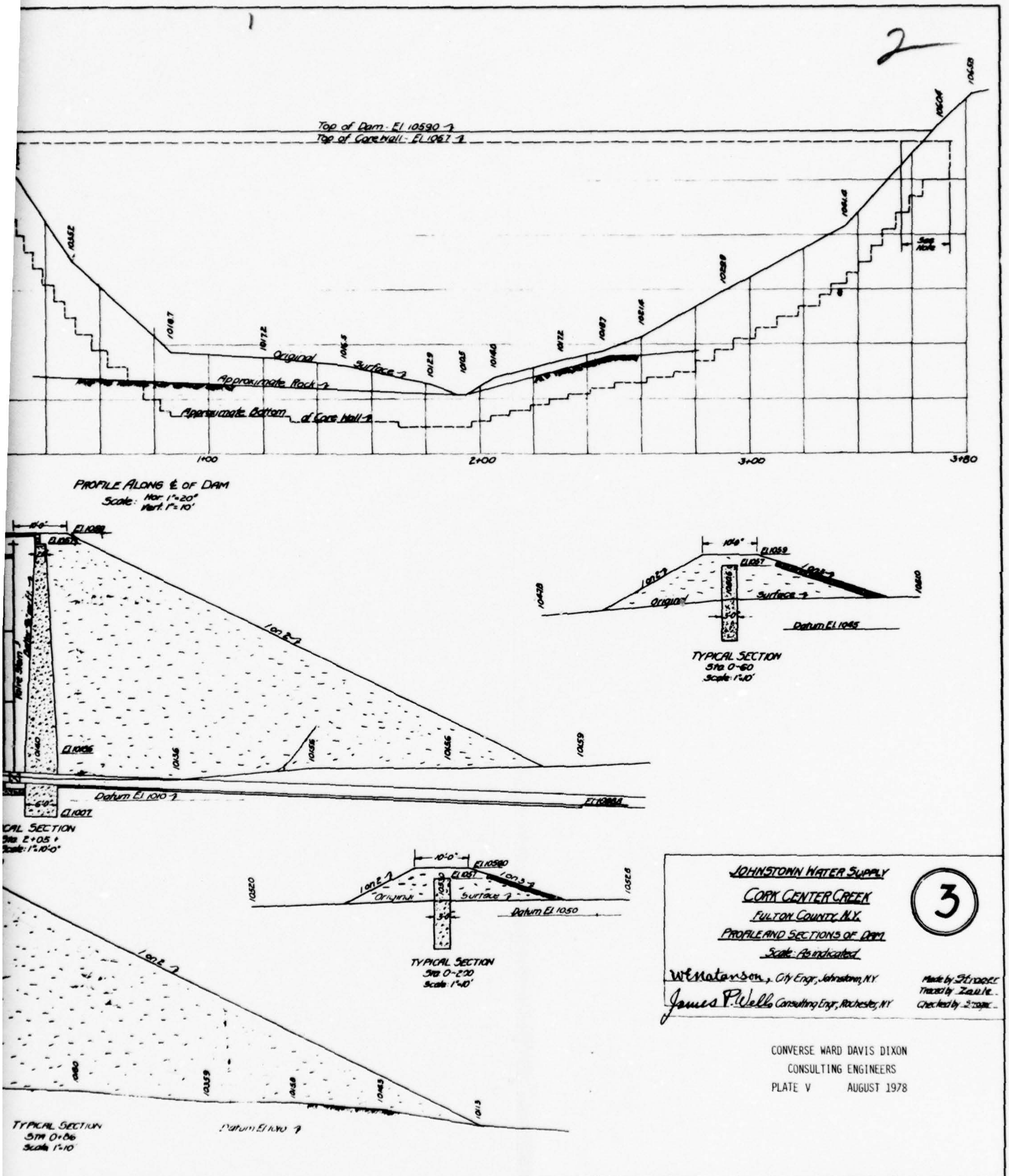

CONVERSE WARD DAVIS DIXON
CONSULTING ENGINEERS
PLATE V AUGUST 1978

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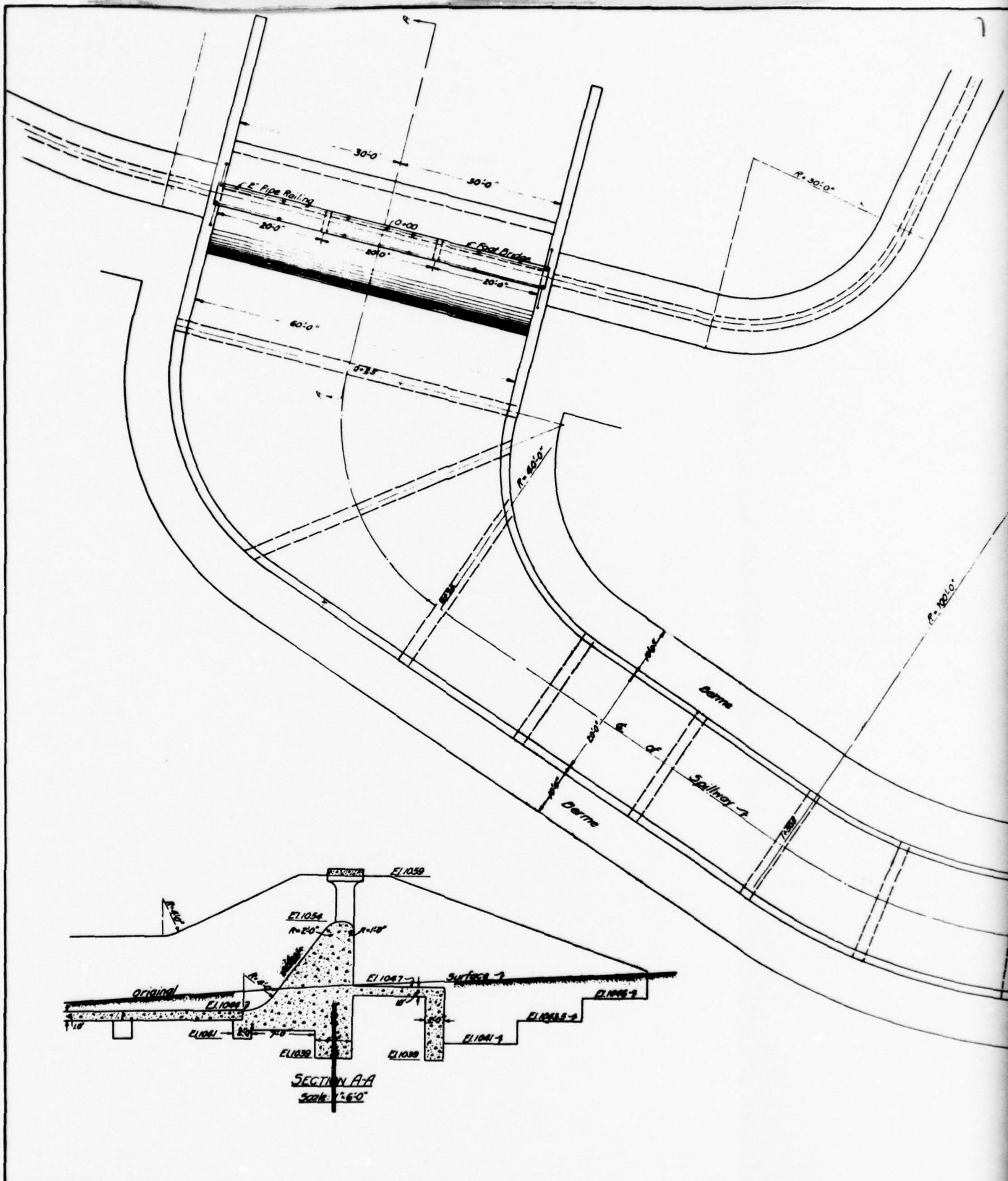

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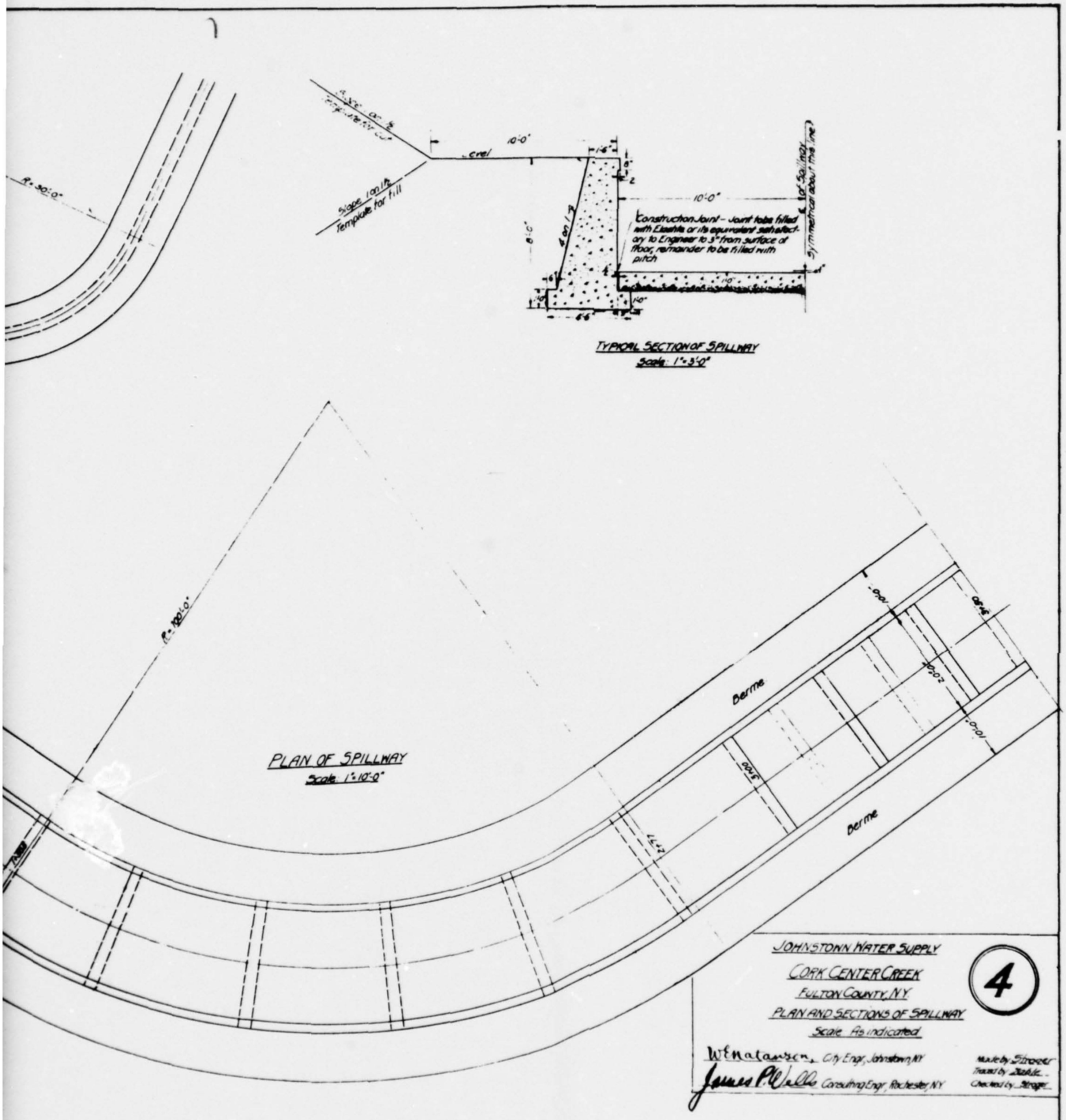

CONVERSE WARD DAVIS DIXON
CONSULTING ENGINEERS
PLATE VI AUGUST 1978

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APPENDIX A

CHECKLIST - ENGINEERING DATA

CHECKLIST

HYDROLOGIC AND HYDRAULIC DATA

ENGINEERING DATA

NAME OF DAM: Cork Center Storage Reservoir Dam NDS ID NO.: NY 658

RATED CAPACITY (ACRE-FEET) 445 NYS DEC ID NO.: 172C-3191

ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): Varies; approx.1054

ELEVATION TOP FLOOD CONTROL POOL (STORAGE CAPACITY): 1055.25

ELEVATION MAXIMUM DESIGN POOL: 1059

ELEVATION TOP DAM: 1059

CREST: (Overflow Spillway):

a. Elevation 1055.25
b. Type Gravity concrete overflow spillway; rounded-crest
c. Width Rounded; approximately 3 feet
d. Length 60 feet
e. Location Spillover Near right end of embankment
f. Number and Type of Gates None

OUTLET WORKS:

a. Type 24-inch C.I. water supply pipe
b. Location Under the embankment; almost along its deepest section
c. Entrance inverts 1043± and 1035± (supply pipe);*
d. Exit inverts 1002±
e. Emergency draindown facilities 24-inch C.I. mud pipe

HYDROMETEOROLOGICAL GAGES:

a. Type None
b. Location None
c. Records None

MAXIMUM NON-DAMAGING DISCHARGE: Unknown; 1650 cfs (estimate)

*1020± (mud pipe)

CHECKLIST

ENGINEERING DATA

DESIGN, CONSTRUCTION, AND OPERATION
PHASE I

NAME OF DAM: Cork Center Storage Reservoir

NDS ID NO.: NY658 NYS DEC ID NO.: 172C-3191

ITEM	REMARKS
DRAWINGS	Plate II: Remodelling details of the spillway crest (1963) Plates III through VI: Plans and cross sections of the embankment and associated structures (1917)
REGIONAL VICINITY MAP	Dam shown (Plate I) on USGS 7½ minute quadrangle sheet of Peck Lake, N.Y. (N43°02'14", W74°27'55")
CONSTRUCTION HISTORY	None available
TYPICAL SECTIONS OF DAM	Sections through earthen embankment and overflow spillway shown on Plates V and VI
HYDROLOGIC/HYDRAULIC DATA	USACE Hydrologic Model for Mohawk River Basin and computed capacity vs pool elevation are the only hydrologic data available. Some hydraulic data available.

ENGINEERING DATA

ITEM	REMARKS
OUTLETS: Plan Details Constraints Discharge Ratings	Changed from what is shown on Plate IV. See Figures 69 and 70 in Appendix E.
RAINFALL/RESERVOIR RECORDS	Records of reservoir water levels available for a certain period extending into the past. No rainfall or any other gages in this basin to our knowledge.
DESIGN REPORTS	Not available
GEOLOGY REPORTS	Very sketchy one-paragraph report (reproduced in Appendix E) by Mr. A. R. McKim, the then inspector of docks and dams for Conservation Commission, State of N.Y.
DESIGN COMPUTATIONS: Hydrology & Hydraulics Dam Stability Seepage Studies	Limited hydrology and hydraulics data and detailed spillway stability analysis (Appendix E). No seepage studies or stability analysis for the earth embankment.

ITEM	REMARKS
MATERIALS INVESTIGATIONS Boring Records Laboratory Field	The reconstruction application (1963) indicates that the embankment is gravel with cobbles and boulders, and the bed is boulders. A letter dated 7-23-17 says natural soil is loamy earth with some boulders. Materials testing of concrete in 1919 indicated greater than 3000 psi strength.
POST-CONSTRUCTION SURVEYS OF DAM	None available
BORROW SOURCES	None available
MONITORING SYSTEMS	None
MODIFICATIONS	24-inch gate valve on the dam crest, 16-inch C.I. bypass pipe on the right of spillway right abutment wall and aeration tank shown on Plate IV were not encountered during field inspection - probably not constructed. In 1963 spillway crest raised by 15 inches from El. 1054 to El. 1055.25; angle-iron access bridge to intake structure constructed. Date of tree planting on downstream slope not known.

ENGINEERING DATA

ITEM	REMARKS
HIGH POOL RECORDS	Maximum observed 2 feet above spillway crest in the early 1970s
POST-CONSTRUCTION ENGINEERING STUDIES AND REPORTS	None
PRIOR ACCIDENTS OR FAILURE OF DAM Description Reports	None reported
MAINTENANCE AND OPERATION RECORDS	None available
SPILLWAY: Plan Sections Details	Plan, elevations and sections in Plates II, V and VI

ENGINEERING DATA

ITEM	REMARKS
OPERATING EQUIPMENT: Plans Details	None available
PREVIOUS INSPECTION Date: Findings	Inspections are performed periodically by NYSDEC. The last inspection report on file is dated 10-23-69. Report, in part, indicates: "Adequate, no apparent repairs needed or minor repairs that can be covered by periodic maintenance."

APPENDIX B

CHECKLIST - VISUAL INSPECTION

CHECKLIST

VISUAL INSPECTION

PHASE I

NAME OF DAM: Cork Center Storage Reservoir Dam   County: Fulton   State: New York   NDS ID No.: NY 658

Keck Creek   NYS DEC ID No.: 172C-3191

Type of Dam: Earthfill-Concrete Core Wall   Hazard Category: High

Date(s) Inspection: 20 July 1978   Weather: Sunny, Warm   Temperature: $80^{o}F$

Pool Elevation at Time of Inspection: 1053.9 msl 16" below top of spillway crest

Tailwater at Time of Inspection: None msl (Below lower gate house)

Inspection Personnel:

E. A. Nowatzki (CWDD)   C. Ackerbauer (City of Johnstown)

G. S. Salzman (CWDD)   R. Lake (City of Johnstown)

T. Newhouse (City of Johnstown)

E. A. Nowatzki   Recorder

Remarks:
Reservoir locally called "High Daddy"

EMBANKMENT

Sheet 1 of 3

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	None visible - animal burrows throughout	
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	None visible	
SLOUGHING OR EROSION: Embankment Slopes Abutment Slopes	None visible for either	
VERTICAL AND HORIZONTAL ALIGNMENT OF THE CREST	Both OK	
RIPRAP FAILURES	None visible - rip-rap extends up to about spillway crest elevation. Vegetation above rip-rap to dam crest.	Woody vegetation on upstream slope above rip-rap should be removed.

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
JUNCTION OF EMBANKMENT WITH: Abutment Spillway Other Features	Embankment with left abutment: Upstream OK. Downstream - moderate seepage starting in crotch about ½ way down slope. Ground wet. Some springs noted along (REFER TO SHEET 3)	
ANY NOTICEABLE SEEPAGE	Seepage noted where downstream embankment meets left abutment (see above). Springs noted. Seepage extends across to toe to right abutment but (REFER TO SHEET 3)	See recommendations in text of the report.
RECORDING INSTRUMENTATION	None	
DRAINS	None	
OTHER	Downstream edge of crest and downstream face heavily wooded with tall pines. Also deciduous trees on downstream slope. 2 large pines on (REFER TO SHEET 3)	Roadway on top of dam in good condition. Some pine roots cross it at surface.

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
JUNCTION OF EMBANKMENT WITH: Abutment Spillway Other Features	crotch and continuing along toe to right embankment crotch. Embankment with right abutment: Upstream OK. Downstream - wet spots in crotch near toe. Embankment with spillway: Shallow embankment sections to left and right of spillway which is located at right angle to main dam at right abutment. No problems evident.	
ANY NOTICEABLE SEEPAGE	does not go up right abutment crotch. Wet zones noted up to about 10-15' vertically above toe. Can hear water running below ground before emergence. Ground wet and spongy. Spring empties into creek channel where 24" supply & mudline pipes empty. Seeping water is clear - no sloughing noted. Decay of vegetation in seepage path indicates it has been going on for some time. No erosion observed.	
OTHER	upstream face near left abutment. Tree growth starting on upstream face above rip-rap line.	

OUTLET WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	Outlet is steel pipe - not visible except at discharge point - looks OK.	
INTAKE STRUCTURE	Access bridge rusted. Gate platform moderately spalled and scaled. Valves for 17' and 25' entry open at all times. Turned and observed (REFER TO SHEET 2)	
OUTLET STRUCTURE	Two 24" pipes go from lake intake structure, under dam, to lower gate house, and then to creek channel. 3 valves in lower gate house, 2 on main (REFER TO SHEET 2)	
OUTLET CHANNEL	Two 24" pipes to creek channel. Steel or iron pipes look OK.	
EMERGENCY GATE	Mud pipe serves as emergency outlet.	

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
INTAKE STRUCTURE	to function OK. 24" mud pipe extends about 30 feet into lake. Valve remains opened at all times. Crack valve OK. Water level inside intake structure at lake level; therefore could not see valves.	
OUTLET STRUCTURE	line - tested & functioning. There was a flow being maintained. Mud pipe valve cracked & functions; effluent very muddy - turned off immediately.	Should have light to valves under wood floor. Valves well maintained - little or no slack.

UNGATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR	Generally OK - Downstream face moderately spalled and eroded.	
APPROACH CHANNEL	Minor spalling and scaling on wing walls.	
DISCHARGE CHANNEL	Minor spalling at wall joints and on wall. Major spall on left wall at lower end of spillway channel. Last floor slab cracked - some seepage (REFER TO SHEET 2)	Seepage is too far below spillway to influence dam.
BRIDGE AND PIERS	Piers moderately eroded (some spalling). Bridge scaled badly. Large spall (exposed steel) about midspan. Other spalls on concrete walkway.	
JUNCTION WITH LEFT WING WALL	Some minor seepage starting about 1/3 of way down to base of spillway as shown by wetness but not flow.	Injection grouting should be done to avoid further deterioration.

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
DISCHARGE CHANNEL	coming from below slab at lower lip.	

INSTRUMENTATION

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS	Downstream flow monitored for water supply purposes.	
OBSERVATION WELLS	None	
WEIRS	Downstream at chlorination house.	
PIEZOMETERS	None	
OTHER	Water level monitored and recorded twice daily at spillway.	

RESERVOIR

Sheet 1 of 1

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	Steep down to water - about 1½ horizontal to 1 vertical. One naturally sandy area covered with moss and scrub, otherwise heavily wooded.	No evidence of slope failure.
SEDIMENTATION	Moderate at upstream entrance. Indication of sedimentation at dam by turbidity of water released from mud pipe.	

DOWNSTREAM CHANNEL

Sheet 1 of 1

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION Obstructions Debris Other	No obstructions or debris. Upper reaches are controlled for chlorination house (water supply channel).	
SLOPES Cover Stability	Generally wooded - appear good.	
APPROXIMATE NUMBER OF HOMES AND POPULATION	One house and barn (Miller Rd) within mile of dam creek. Trailer off Wemple Rd (about 1½ miles downstream) + 2 houses in Keck's Center (about 2 miles downstream).	Concur with high hazard designation.

APPENDIX C

COMPUTATIONS

BY J.K. DATE 8/7/78 | **JOSEPH S. WARD** 91 ROSELAND AVE. CALDWELL, N. J. | SHEET NO. 1 OF 13

CHKD. BY [illegible] DATE [illegible] | JOB NO. A7805-11 [illegible]

SUBJECT Cork Center Storage Reservoir – Hydrology

Classification of Dam

Maximum height of dam = 40 ft.

49 ft from sheet #3 (Plate 5) of Johnstown Water Supply

Storage at spillcrest elevation (1055.25′) = 145,000,000 gallons

= 445 acre ft.

Information obtained from Oct. 7, 1963 application for raising Cork Center storage reservoir spillway crest

The dam is, therefore, 'intermediate size' based on its height as determined from the Department of the Army Office of the Chief Engineer's "Recommended guidelines for Safety Inspection of Dams." Since the dam has been classified as a high hazard dam, the forementioned publication requires the spillway to be designed for passing the probable maximum flood (PMF)

Flow over spillway when pool is at the top of dam i.e. El. 1059.0′.

Effective length of spillway crest $L = L' - 2(NK_p + K_a)H_E$

$L' = 58'$

$N = 2$

$K_p = 0.02$

$K_a = 0.0$

$H_e = 1059 - 1055.25 = 3.75$ ft.

Bureau of Reclamation Design of Small Dams Page 373

$\therefore L = 58 - 2(2 \times .02 + 0)3.75 = 57.7$ ft.

$$Q = C_o L H_o^{3/2}$$

Assume the crest to behave as ogee crest, Then from page 378 of Bureau's Design of Small Dam

for $P = 7.25$ ft., $H_o = 3.75$ ft. $\frac{P}{H_o} = \frac{7.25}{3.75} = 1.93$

and $C_o = 3.94$

$$\therefore Q = 3.94 \times 57.7 \times (3.75)^{3/2} = 1650 \text{ cfs}$$

From page 97, 98, 100, 106 and 107 of USACE Model study of Upper Hudson & Mohawk River Basins

Use sub-basin 22 of the Mohawk basin, Little falls, N.Y. to Mouth

Subarea	Area (mi^2) A_1	SPF_1	TA (Transposed Agnes)
22	23	10655 cfs	7538 cfs

Since SPF is greater than TA the SDF will be based on 2 x SPF = PMF

Drainage Area of Cork Center Reservoir $= A_2 = 2.6$ mi^2 {Oct. 14, 1963 approx. for raising spillway crest}

$$\left(\frac{A_1}{A_2}\right)^{0.75} = \left(\frac{F_1}{F_2}\right)$$

$$\left(\frac{23}{2.6}\right)^{0.75} = \left(\frac{10655}{F_2}\right) \quad \text{or } F_2 = \frac{10655}{5.13} = 2077 \text{ cfs}$$

$$PMF = 2\ SPF = 2 \times 2077 = 4154 \text{ cfs}$$

Assume Max. pool elevation at top of dam = 1059′ which is 3.75 ft above normal pool at spillway crest (El. 1055.25′)

General slope of the shore line = $1\frac{1}{2}H : 1V$ ✓ (Field inspection)

Length of shore line = 1.78 miles (from USGS quad.) ✓

Lake size at normal pool elevation = 41 acres ✓ (Oct. 14, 1963 appl...

Volume change from normal pool to maximum pool = ΔV =

$$= (1.5 \times 3.75) \times \frac{1}{2} \times 3.75 \times 1.78 \times 5280 \times \frac{1}{43560} + (3.75 \times 41)$$

$$= 2.28 + 153.75 = 156 \text{ acre feet.}$$

∴ storage capacity at max. pool = storage capacity at normal pool + ΔV

$$= 445 + 156 = 601 \text{ acre feet.}$$

Overtopping Potential

Q_1 = Max. spillway capacity = 1650 cfs

Q_2 = PMF = 4154 cfs

p_{PMF} = % of PMF spillway will pass; % of SPF spillway will pass

or $p_{PMF} = \frac{Q_1}{Q_2} = \frac{1650}{4154} = 40\%$ $\quad p_{SPF} = \frac{1650}{2077} = 79\%$

$$\therefore (1-p) = \frac{\text{Required reservoir storage}}{\text{Vol. of inflow hydrograph}} = 0.60 \text{ (for PMF)}$$

$$= 0.21 \text{ (for SPF)}$$

Solving for Tp for triangular hydrograph. Assume Tp is a function of the linear elements of equivalent areas

$A_1 = 23 = \frac{\pi}{4} d_1^2$ $\qquad Tp_1 = 9$ hr. → Upper Hudson and Mohawk river basins flood routing models

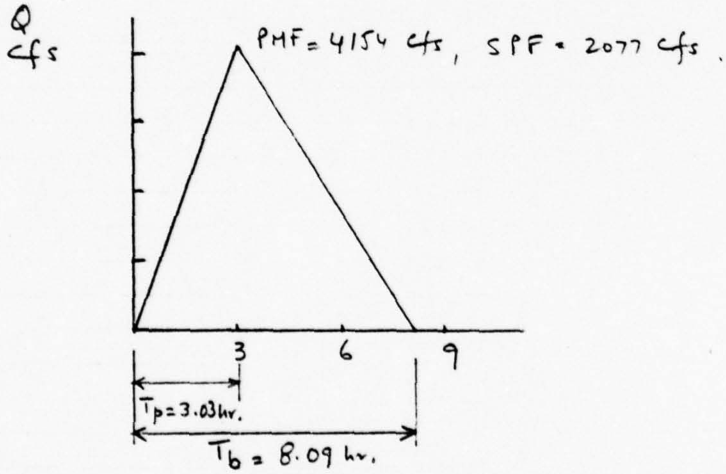
or $d_1 = 5.41$

$A_2 = 2.6 = \frac{\pi}{4} d_2^2$

or $d_2 = 1.82$ $\qquad T_{P_2} = ?$

$$Tp_2 = \frac{d_2}{d_1} Tp_1 = \frac{1.82}{5.41} \times 9 = 3.03 \text{ hrs.}$$

$$Tb_2 = 2.67 (T_{P_2}) = 2.67 \times 3.03 = 8.09 \text{ hrs.}$$ (Design of small dams page 69



Volume of inflow hydrograph = V = $\frac{PMF \times T_b}{2}$

For PMF $= \frac{1}{2} \times 4154 \times 8.09 \times \frac{3600}{43560} = 1389$ acre ft.

For SPF $= 695$ acre ft.

Required reservoir storage = 0.6 x 1389 = 833 acre ft for PMF
= 0.21 x 695 = 146 acre ft. for SPF

But available incremental storage = 156 acre ft. (sheet 3)

156 < 833 acre ft. (PMF)   156 > 146 (SPF)

∴ CORK CENTER RESERVOIR WILL NOT BE ABLE TO CONTAIN THE PMF WITHOUT OVERTOPPING OF THE DAM — BUT WILL BE ABLE TO CONTAIN THE SPF WITHOUT OVERTOPPING. However, more accurate analysis follows in the succeeding pages.

JOSEPH S. WARD
91 ROSELAND AVE. CALDWELL, N. J.

BY J. K. DATE 8/7/78 SHEET NO. 6 OF 13
CHKD. BY PGM DATE 8/8/78 JOB NO. A7805-11F
SUBJECT Cork Center Storage Reservoir - Hydrology

Discharge Q (cfs) vs head over spillway crest.

p = 7.25

$$Q = C_o L H_o^{3/2}$$

H_o	EL.	Q		cfs	
0.5	1055.75	$= 3.95 \times 57.7\,(0.5)^{3/2}$	=	81	
1.0	1056.25	$= 3.95 \times 57.7\,(1)^{3/2}$	=	228	
1.5	1056.75	$= 3.95 \times 57.7\,(1.5)^{3/2}$	=	419	
2.0	1057.25	$= 3.95 \times 57.7\,(2)^{3/2}$	=	645	
2.5	1057.75	$= 3.95 \times 57.7\,(2.5)^{3/2}$	=	901	
3.0	1058.25	$= 3.945 \times 57.7\,(3)^{3/2}$	=	1183	
3.5	1058.75	$= 3.94 \times 57.7\,(3.5)^{3/2}$	=	1489	
3.75	1059.00	$= 3.935 \times 57.7\,(3.75)^{3/2}$	=	1649	← Max. pool
4.0	1059.25	$= 3.93 \times 57.7\,(4)^{3/2}$	=	1814	
4.5	1059.75	$= 3.925 \times 57.7\,(4.5)^{3/2}$	=	2162	
5.0	1060.25	$= 3.92 \times 57.7\,(5)^{3/2}$	=	2529	
5.5	1060.75	$= 3.91 \times 57.7\,(5.5)^{3/2}$	=	2910	
6.0	1061.25	$= 3.905 \times 57.7\,(6)^{3/2}$	=	3311	

BY J.K. DATE 8/7/78 | JOSEPH S. WARD 91 ROSELAND AVE. CALDWELL, N. J. | SHEET NO. 7 OF 13

CHKD. BY RGM DATE 8/8/78 | JOB NO. A 7805-11 E

SUBJECT Cork Center Storage Reservoir – Hydrology

Flood storage vs head above crest or elevation

Lake Area at normal pool elevation = 41 acre
General slope of shore line = $1\frac{1}{2}$ H : 1 V
} sheet

Head H	EL.	Vol. = $(H)(41) + \left[\frac{(H)(1.5H)}{2} \times \text{Length of shore} \times \frac{5280}{43560}\right]$ (Length of shore = 1.78)		
0.5'	1055.75	= 0.5x41 + 0.04	= 21	acre ft.
1.0'	1056.25	= 41 + 0.16	= 41	
1.5	1056.75	= 61.5 + 0.36	= 62	
2.0	1057.25	= 82 + 0.65	= 83	
2.5	1057.75	= 102.5 + 1.0	= 104	
3.0	1058.25	= 123 + 1.46	= 124	
3.5	1058.75	= 143.5 + 1.98	= 145	
3.75	1059.00	= 153.8 + 2.28	= 156	← Max. pool
4.0	1059.25	= 164 + 2.59	= 167	
4.5	1059.75	= 184.5 + 3.28	= 188	
5.0	1060.25	= 205 + 4.0	= 209	
5.5	1060.75	= 225.5 + 4.9	= 230	
6.0	1061.25	= 246 + 5.8	= 252	

BY J. K. DATE 8/7/78

JOSEPH S. WARD
91 ROSELAND AVE. CALDWELL, N. J.

SHEET NO. 8 OF 13

CHKD. BY RGm DATE 8/3/78

JOB NO. A7805-11 F

SUBJECT Cork Center Storage Reservoir – Hydrology

EL. (ft.)	Q (cfs)	Q/2 (cfs)	Flood Storage acre-ft.	S Flood storage cfs-hrs.	S/ΔT (0.5 hr)	$SI = \frac{Q}{2} + \frac{S}{\Delta}$
Normal pool 1055.25	0	0	0	0	0	0
1055.75	81	40	21	254	508	548
1056.25	228	114	41	496	992	1106
1056.75	419	210	62	750	1500	1710
1057.25	645	323	83	1004	2008	2331
1057.75	901	451	104	1258	2516	2967
1058.25	1183	592	124	1500	3000	3592
1058.75	1489	745	145	1755	3510	4255
1059.00	1649	825	156	1888	3776	4601
1059.25	1814	907	167	2021	4042	4949
1059.75	2162	1081	188	2275	4550	5631
1060.25	2529	1265	209	2529	5058	6323
1060.75	2910	1455	230	2783	5566	7021
1061.25	3311	1655	252	3049	6098	7753

BY J.K. DATE 8/7/78

CHKD. BY [illegible] DATE 8/7/78

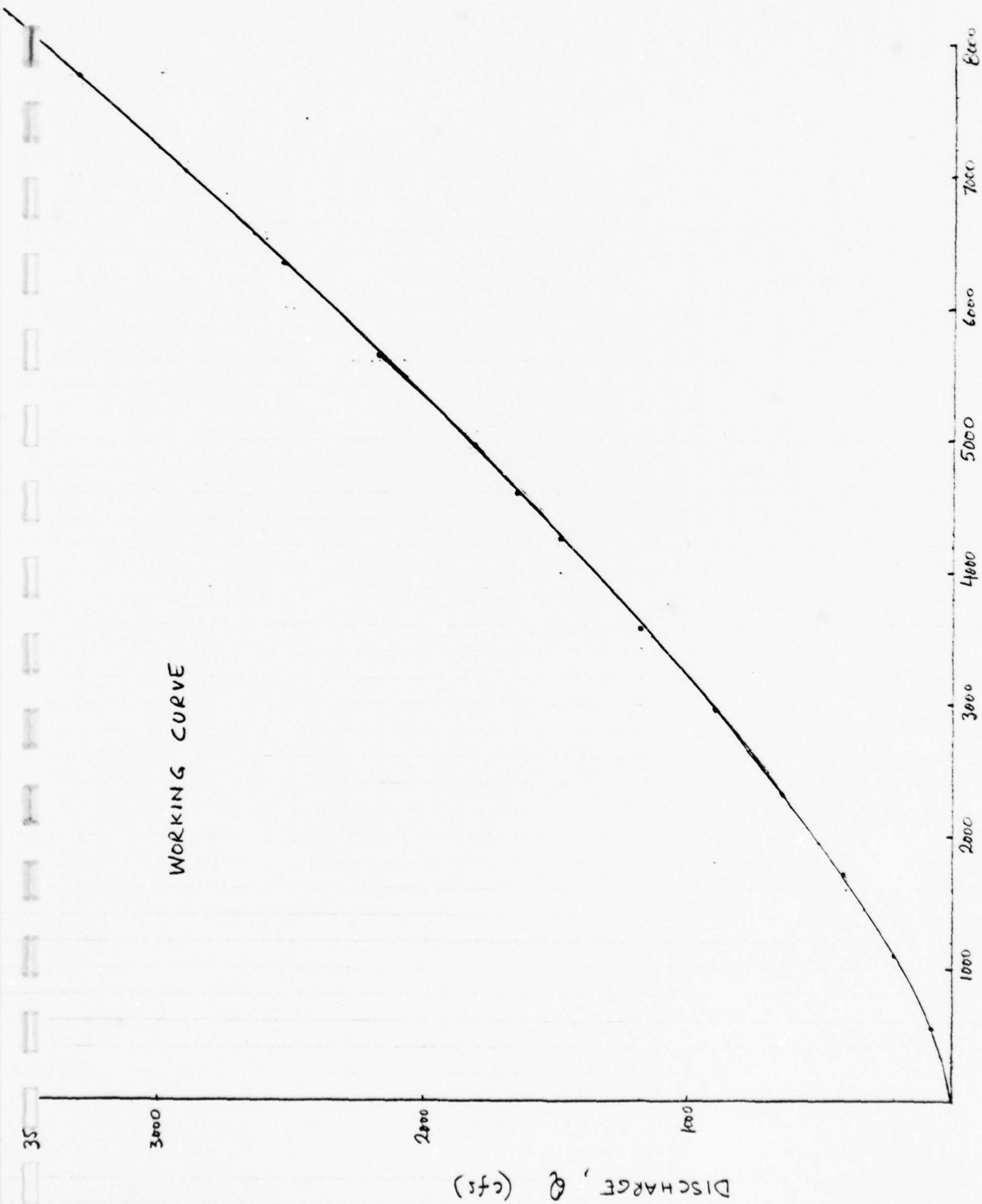
SUBJECT Cork Center Storage Reservoir – Hydrology

JOSEPH S. WARD

91 ROSELAND AVE. CALDWELL, N. J.

SHEET NO. 9 OF 13

JOB NO. A7805-11F



BY J.K. DATE 8/7/78 — JOSEPH S. WARD, 91 ROSELAND AVE. CALDWELL, N. J. — SHEET NO. 10 OF 13

CHKD. BY P.B.M. DATE 8/8/78 — JOB NO. A7805-11F

SUBJECT Cork Center Storage Reservoir – Hydrology

Time (Hrs.)	I (cfs)	$\bar{I}$	$(SI)_N$	Q	$(SI)_{N+1} = (SI)_N - Q_N + \bar{I}_{(N+1)}$
0	0	0	0	0	0 − 0 + 335 = 335
0.5	670	335	335	40	335 − 40 + 1005 = 1300
1.0	1340	1005	1300	280	1300 − 280 + 1685 = 2705
1.5	2030	1685	2705	800	2705 − 800 + 2375 = 4280
2.0	2720	2375	4280	1470	4280 − 1470 + 3065 = 5875
2.5	3410	3065	5875	2200	5875 − 2200 + 3755 = 7430
3.0	4100	3755	7430	3160	7430 − 3160 + 3930 = 8200
3.5	3760	3930	8200	3590	8200 − 3590 + 3555 = 8165
4.0	3350	3555	8165	3560	8165 − 3560 + 3145 = 7750
4.5	2940	3145	7750	3300	7750 − 3230 + 2740 = 7260
5.0	2540	2740	7190	3000	7260 − 3050 + 2335 = 6545
5.5	2130	2335	6525	2640	6545 − 2670 + 1925 = 5800
6.0	1720	1925	5810	2260	5800 − 2260 + 1520 = 5060
6.5	1320	1520	5070	1870	5060 − 1850 + 1110 = 4320
7.0	900	1110	4310	1500	4320 − 1470 + 700 = 3550
7.5	500	700	3510	1140	3550 − 1140 + 295 = 2705
8.0	90	295	2665	770	2705 − 800 + 45 = 1950
8.5	0	45	1940	500	1950 − 500 + 0 = 1440

BY J.K. DATE 8/7/78

JOSEPH S. WARD

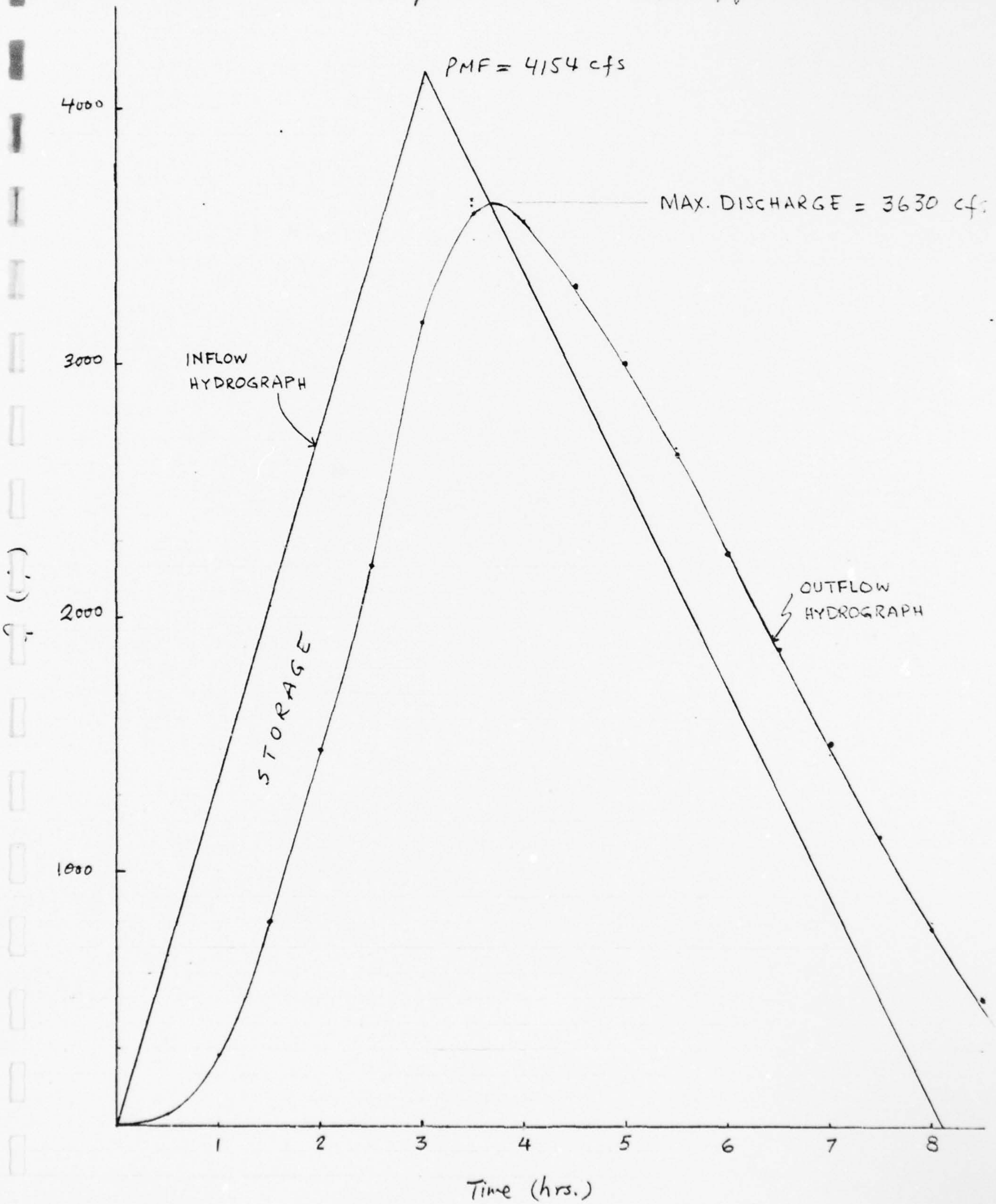
91 ROSELAND AVE. CALDWELL, N. J.

SHEET NO. 11 OF 13

CHKD. BY PLM DATE 9/5/78

JOB NO. A7805-11F

SUBJECT Cork Center Storage Reservoir – Hydrology



From outflow hydrograph, discharge at maximum pool = 3630 cfs

Determine H by trial and error

$Q = C_o L H_o^{3/2}$

For $H_o = 6.4$ ft

$Q = 3.90 \times 57.7 (6.4)^{3/2} = 3643$ cfs , close enough

∴ The PMF will raise the pool to 1055.25 + 6.4 = 1061.65'

El. 1061.65' is 2.65' above the top of the dam.

% OF PMF THAT CAN BE PASSED AT PEAK OUTFLOW IS:

$$\frac{\text{MAX OUTFLOW w/ POOL ELEV. AT DAM CREST}}{\text{MAX. DISCHARGE FOR PMF INFLOW}} = \frac{1650 \text{ cfs}}{3630 \text{ cfs}} \times 100 = 45\%$$

<u>Discharge through supply and mud pipes</u>

upstream head on mud pipe = 40 feet
" " " supply line = 25 feet (minimum)

From original drawings the difference in upstream and downstream water levels = 1059 − 1002 = 57 feet.

Since the actual invert level at the outfall and the length of pipes are not exactly known, the total head loss due to entrance, through gate valves, bends, and friction in pipes may be conservatively taken as 17 feet and discharge will be computed for 40 feet head.

$$Q = AV = A\sqrt{2gh} = \frac{\pi}{4}(2)^2\sqrt{2\times 32.2\times 40}$$

$$= 159 \text{ cfs}$$

$\therefore$ discharge through 2 pipes = 320 cfs.

If the flow through these two pipes is taken into consideration, the % of PMF that can be passed at peak outflow $= \frac{1650+320}{3630} = 54\%$.

The original calculations in the file are quite elaborate and deal in depth with items of even minor significance. A rough check of the original analysis revealed it to be in conformance with standard engineering practice except in one case (page 28) where active earth pressure coefficient was used for calculating passive resistance.

Present analyses are performed on the section of the section of the dam shown on the next sheet.

(1) Stability of the spillway against overturning about its toe

Resisting forces and moments.

Weight of the concrete section = $(2.5 \times 12)140 + (\frac{1}{2} \times 9 \times 12)140 + (3 \times 4)140$

$= 4200 + 7560 + 1680 = 13440$

Moment about pt. P $= (4200 \times 10.25) + (7560 \times 9 \times \frac{2}{3}) + (1680 \times 9.5)$

$= 43050 + 45360 + 15960 = 104370$ ft-lb

Wt. of triangular section of water over the crest

$= \frac{1}{2} \times 4 \times 2.5 \times 62.4 = 312$ lb.

Moment about toe $= 312\,(2/3 \times 2.5 + 9) = 3328$ ft-lb.

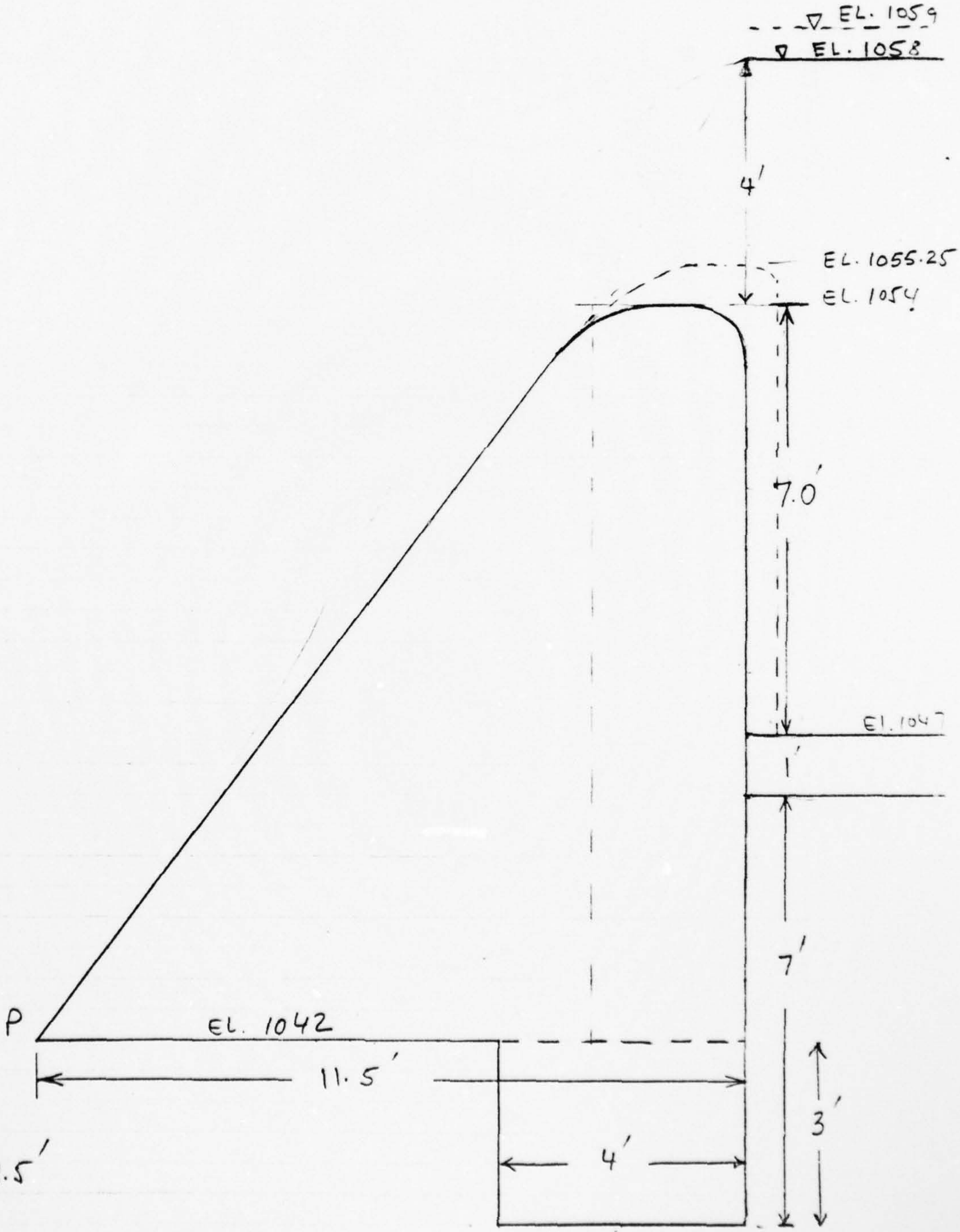
Neglect the active forces on the u/s and D/s faces of the cut-off wall because their net result is small.

Therefore total resisting moment $= 104370 + 3328 = 107700$ ft-lb

BY J.K. DATE 8/3/78 — JOSEPH S. WARD — SHEET NO. 2 OF 10

CHKD. BY HDD DATE 8/4/78 — 91 ROSELAND AVE. CALDWELL, N. J. — JOB NO. A7805-11F

SUBJECT Cork Center Storage Reservoir spillway stability



Scale 1" = 2.5'

Overturnining Forces and Moments

Horizontal water thrust above El. 1047

$$= \frac{62.4}{2}\left[(11)^2-(4)^2\right] = 3276 \text{ lb.}$$

Moment about toe = 3276 × 8 = 26208 ft-lb

Assuming head of water 7' and 6' at El. 1046' and 1042' respective

This adds to the overturning moment.

$$\text{Horizontal Thrust} = \frac{62.4}{2}\left[\left(\frac{7+6}{2}\right)\times 4\right] = 811 \text{ lb.}$$

Moment = 811 × 2 = 1622 ft-lb.

Neglect active earth pressure between El. 1046 + 1042

Uplift pressures

The upstream wall below El. 1047 (Drawing #4, Dm 427M), the u/s slab and cut-off wall should be effective in reducing the uplift pressure. Assuming that uplift varies from a head of 5.5 ft at the U/s end of the dam base to zero at the downstream end.

Hence total uplift pressure $= \frac{1}{2} \times 5.5 \times 11.5 \times 62.4 = 1973$

Moment about toe $= 1973 \times 11.5 \times \frac{2}{3} = 15126$ ft-lb.

Net vertical load = 13440 + 312 − 1973 = 11779 lb.

If $\bar{x}$ is the distance of the resultant from the toe then

$$11779\,\bar{x} = 107700 - 26208 - 1622 - 15126$$

$$\text{or } \bar{x} = \frac{64744}{11779} = 5.5 \text{ ft.}$$

$$1/3\text{rd base width} = \frac{11.5}{3} = 3.8 \text{ ft.}$$

The resultant falls within the middle third of the base width of the spillway and therefore safe against overturning.

Stability with respect to sliding:

Passive resistance to the cut off wall not considering the weight of the dam $= \frac{(100-62.4)}{2} (3)^2 \tan^2(45+ 33/2)$

$= 574$ lb.

The same amount of passive resistance will be developed by the keying effect of the toe. Therefore, total passive resistance = 1148 lb.

Assume $\mu = 0.3$

Total resistance $= (0.3 \times 11779) + 1148 = 4682$

Driving forces $= 3276 + 811 = 4087$

F.S. against sliding $= \frac{4682}{4087} = 1.15$

The calculations in the original file indicated the spillway to be unstable with respect to sliding. This was because of two main differences in the two analyses. (1) The original analysis assumed an open joint between the horizontal slab and the vertical upstream face of the spillway and, therefore, a higher intensity of hydrostatic pressure was taken

along the vertical upstream side of the section and consequently at the base of the dam also.

(2) No passive resistance to cut off wall and at the toe was considered.

(2) <u>Checking the stability of the spillway with water at the top of the dam ie at El. 1059</u>

Resisting forces and moments will practically remain the same.

Horizontal Water pressure above El. 1047

$$= \frac{62.4}{2}\left[(12)^2 - (5)^2\right] = 3713 \text{ lb.}$$

Moment about toe $= 3713 \times 8 = 29704$ ft-lb.

Horizontal pressure and moment below the upstream slab will hardly be affected and are, therefore, taken the same i.e. 811 lb. and 1622 ft-lb respectively.

Uplift pressure & moment will also remain unchanged. i.e. 1973 lb. and 15126 ft-lb.

$$\text{Hence} \quad 11779\,\bar{x} = 107700 - 29704 - 1622 - 15126$$

$$\text{or } \bar{x} = \frac{61248}{11779} = 5.2 \text{ ft.} > 3.8 \text{ ft}$$

The spillway is still safe against overturning

$$\text{F.S. against sliding} = \frac{4682}{3713 + 811} = 1.03$$

which is just safe against sliding.

BY J.K. DATE 8/16/78 | JOSEPH S. WARD | SHEET NO. 6 OF 10

CHKD. BY P.M. DATE 8/23/78 | 91 ROSELAND AVE. CALDWELL, N. J. | JOB NO. A7805-11F

SUBJECT Cork Center Storage Reservoir Spillway Stability

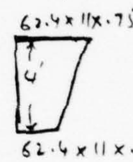
(3) Revising stability calculations for the remodelled section of the dam.

Water at El. 1059

Horizontal water pressure $= \frac{62.4}{2}\left[(11)^2 - (3.75)^2\right] = 3336$ lb.

Horizontal water pressure on the vertical u/s face of the wall below the u/s slab $= \left(\frac{62.4 \times 11 \times .75 + 62.4 \times 11 \times .5}{2}\right) 4$

$= 1716$ lb.



∴ Total driving force $= 3336 + 1716 = 5052$ lb

Passive resistance from the two keys = 1148 lb (page 4)

Passive resistance taking into account the weight of the dam $= \frac{13440}{11.5} \times \underbrace{3}_{=Kp} \times \underbrace{3}_{\text{depth of key}} = 10518$ lb/ft. (discounting the nominal weight of water over the crest)

wt. of water on the upstream slab less 75% uplift

$= \left[11 - (.75 \times 11)\right] \times 8.83 \times 62.4 = 1515$ lb.

Wt. of concrete on the upstream slab $= 140\left[(1 \times 9.25) + (2 \times 8.83)\right]$

$= 3767$ lb.

wt. of concrete in the gravity section = 13440 lb (Page 1; neglecting the small amount of additional concrete on the crest)

Assuming 50% uplift on the base of the dam = 1973 lb (Page

∴ Net downward load $= 1515 + 3767 + 13440 - 1973 = 16749$ lb.

∴ F.S. against sliding $= \frac{(16749 \times .3) + 1148 + 10518}{5052} = 3.3$

stability against overturning for water at top of dam will not be checked because the original section was very safe (page 3 & 4) and the new section has got even more weight added to it.

(4) <u>Checking stability of the remodelled section of the dam with water at El. 1055.25 (dam crest) and ice thrust of 4k per linear foot at El. 1054.25</u>

<u>stability against overturning</u>:

Resisting moment from page 1 = 104370 ft-lb.

Moment due to weight of the additional concrete in the remodelled section $= (1 \times 8.25) \times 140 \times 12 = 13860$ ft-lb.

Total resisting moment $= 104370 + 13860 = 118230$ ft-lb.

<u>Overturning loads & moments</u>:

Horizontal water pressure $= \frac{62.4}{2}(7.25)^2 = 1640$ lb.

moment $= 1640 \times 8.417 = 13803$ ft-lb.

Horizontal pressure on the vertical upstream face of the spillway below elevation 1046 $= \left(\frac{\overset{340}{62.4 \times 7.25 \times .75} + 62.4 \times 7.25 \times .5}{2}\right) 4$

$= 1131$ lb.

Its moment $= 1131 \times 2 = 2262$ ft-lb.

Assuming 50 % uplift $= \frac{1}{2} \times (7.5 \times .5) \times 11.5 \times 62.4 = 1346$ lb.

Its moment $= 1346 \times 11.5 \times 2/3 = 10319$ ft-lb.

Moment due to ice Thrust = 4^{k} x 12.25 = 49000 ft. lb.

Total overturning moments = 13803 + 2262 + 10,319 + 49,000

= 75,384 ft. lb.

Net downward vertical load = 13,440 + (8.25x140) − 1346

= 13249 lb.

∴ point of application of the resultant $\bar{x} = \frac{118,230 - 75,384}{13249}$

= 3.2

1/3rd base width = $\frac{11.5}{3}$ = 3.8' > 3.2' ∴ unsafe

Applying ice thrust 3' lower then $\bar{x} = \frac{118230 - 75384 + 12000}{13249}$ = 4.1' ∴ OK

However, the stabilising effect of the weight of water over the U/S slab has not been considered in the analysis, which will bring the resultant in the middle third of the base. As a precaution during winter, water level should be kept at least 3 ft. below crest level to reduce the overturning moment due to ice.

Stability against sliding;

Driving forces = 1640 + 1131 + 4000 = 6771 lb.

Passive resistance from page 6 = 1148 + 10518 = 11666

Weight of water on the upstream slab less 75% uplift

= [7.25 − (.75x 7.25)] x 8.83 x 62.4 = 1000 lb.

Wt. of concrete on the upstream slab = 3767 lb (page 6)

∴ Net vertical downward load = 1000 + 3767 + 13440 − 1346 =

$= 16861$ lb.

$\therefore$ F.S. against sliding $= \dfrac{(16861 \times 0.3) + 11666}{6771} = 2.5$

BY J.K. DATE 8/17/78 **JOSEPH S. WARD** SHEET NO. 10 OF 10
CHKD. BY PEM DATE 8/28/78 91 ROSELAND AVE. CALDWELL, N. J. JOB NO. A7805-11F
SUBJECT Cork Center Storage Reservoir spillway stability

SUMMARY OF STABILITY ANALYSIS

STRUCTURE ANALYZED	ASSUMPTIONS	SAFETY AGAINST OVERTURNING	F.S. (SLIDING)
1) Original shape before 1963	(a) No passive resistance to u/s key from the weight of the dam (b) No sliding resistance from u/s slab. (c) 50% uplift on the base of the dam. (d) U/s water level at El. 1058	Resultant almost in the middle of the base. Therefore, ok.	1.15
2) Original shape before 1963	(a), (b) & c, same as above (d) U/s water level at El. 1059	Resultant in the middle third. Therefore ok.	1.03
3) Remodelled section of the spillway in its existing shape	(a) Passive resistance to u/s key from the weight of the dam considered. (b) Sliding resistance from u/s slab accounted for c) 75% uplift under the u/s slab and 50% uplift on the base of the dam d) U/s water level at El. 1059	Did not analyze because it will not be much different from the first two conditions. Therefore ok.	3.3
4) Remodelled section of the spillway ie in its present shape	(a), (b) and c, same as in # 3 above. (d) U/s water level at El. 1055.25 (e) ice thrust of 4 k/ft. of dam at El. 1054.25	Resultant outside the middle third. Unsafe. During winter, keep pool level at least 3 ft. below spillway crest.	2.5

APPENDIX D

PHOTOGRAPHS

FIGURE 1 CREST OF DAM LOOKING LEFT

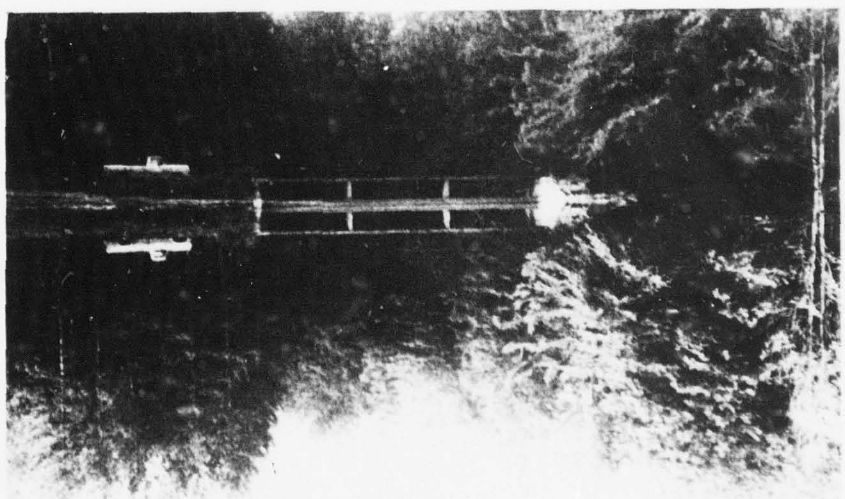


FIGURE 2 SPILLWAY OVERVIEW

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NATIONAL DAM SAFETY PROGRAM. CORK CENTER STORAGE RESERVOIR DAM --ETC(U)
SEP 78 G S SALZMAN DACW51-78-C-0035

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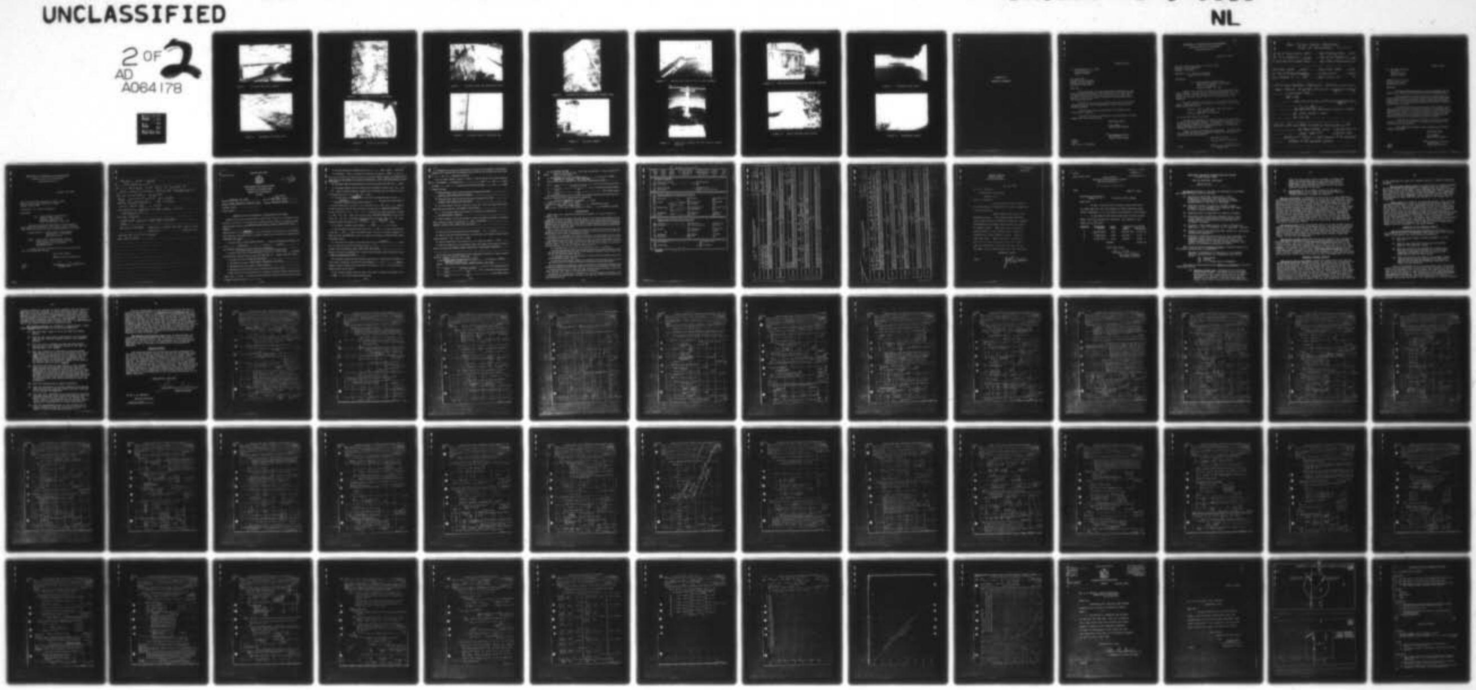

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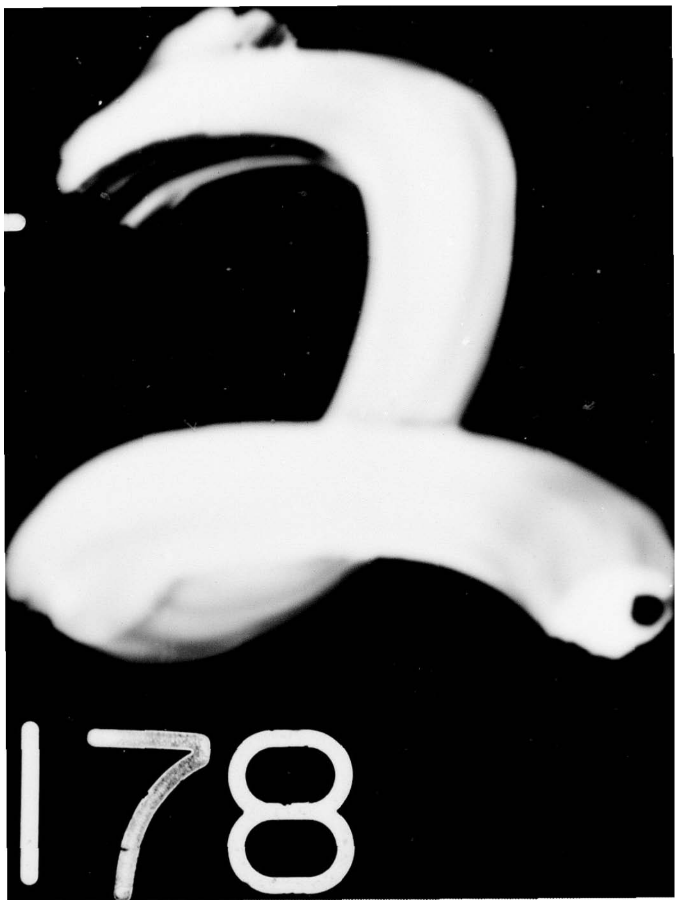
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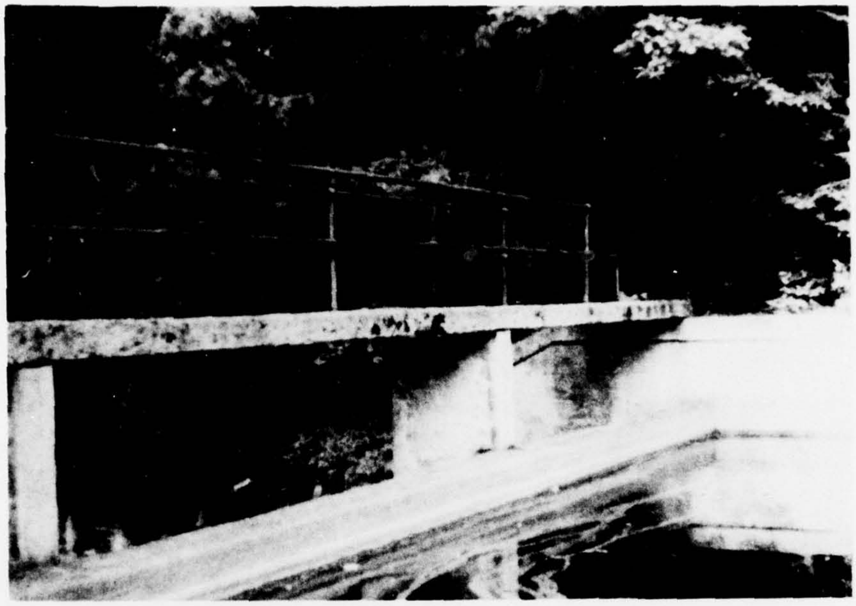


FIGURE 3 SPILLWAY AND RIGHT WINGWALL

FIGURE 4 EMBANKMENT UPSTREAM SLOPE

FIGURE 5 SEEPAGE AT TOE OF EMBANKMENT

FIGURE 6 DECAY OF VEGETATION

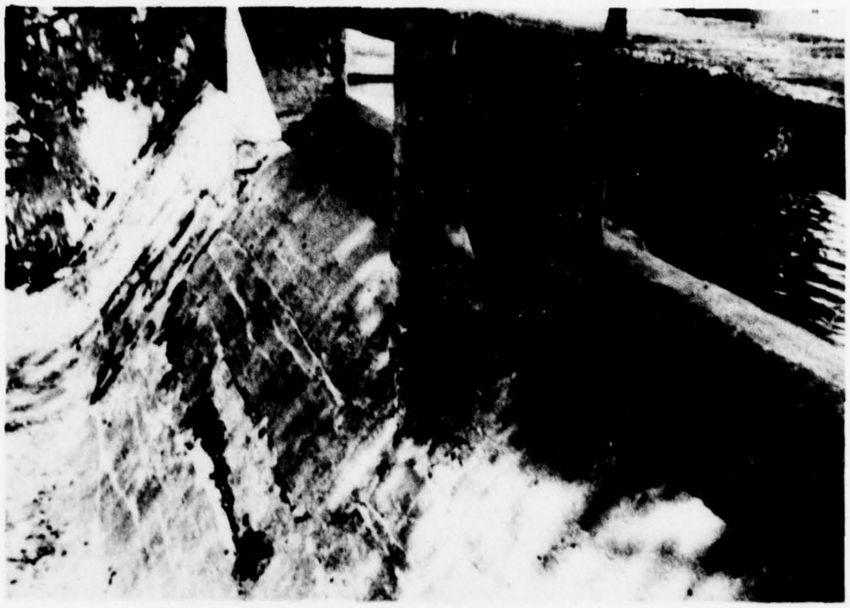


FIGURE 7 SPILLWAY PIERS AND DOWNSTREAM SLOPE

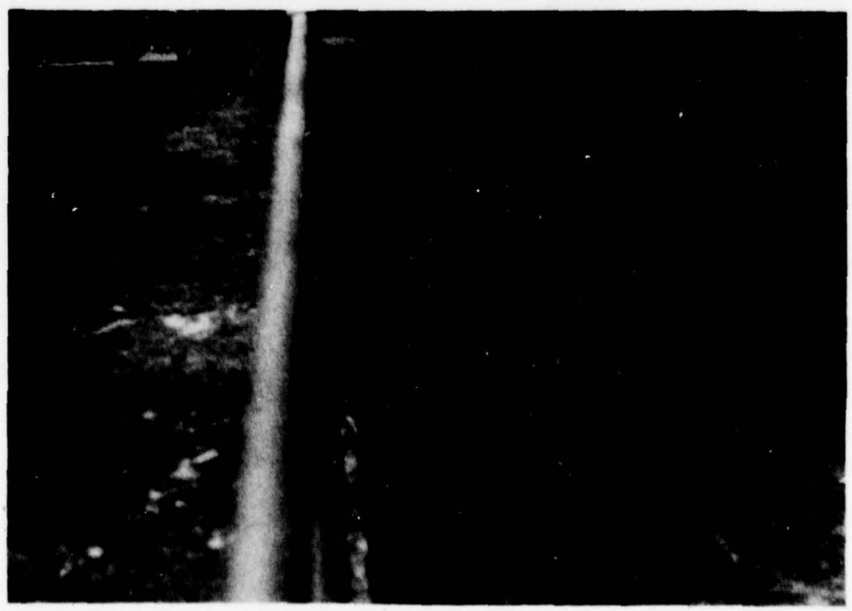


FIGURE 8 SPILLWAY AND LEFT ABUTMENT WALL

FIGURE 9 CLOSE-UP OF SPILLWAY AND LEFT ABUTMENT WALL

FIGURE 10 SPILLWAY CHANNEL

FIGURE 11 CRACKED LAST SLAB OF THE SPILLWAY CHANNEL

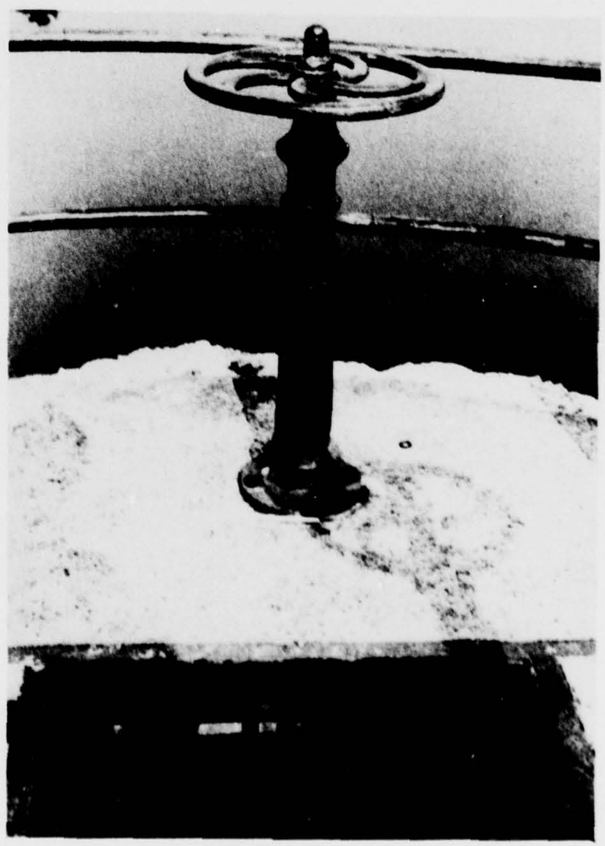


FIGURE 12 INTAKE BOX ENTRANCE AND GATE STEM AT INTAKE STRUCTURE

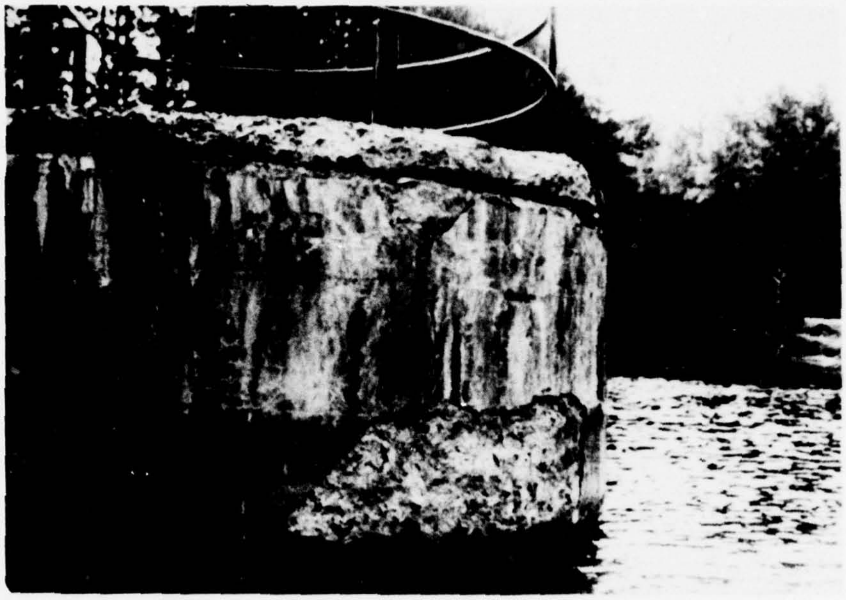


FIGURE 13 SPALLING AND SCALING OF THE INTAKE STRUCTURE

FIGURE 14 SUPPLY AND MUD PIPES OUTFALL

FIGURE 15   RESERVOIR SIDE SLOPES

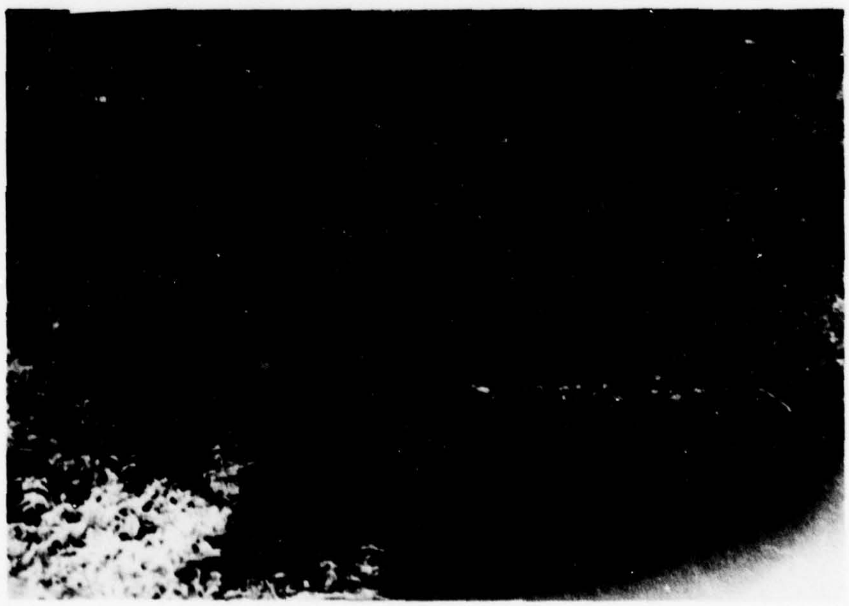


FIGURE 16   DOWNSTREAM CHANNEL

APPENDIX E

RELATED DOCUMENTS

October 17, 1963

Re: Reconstruction of Dam #127
Town of Johnstown
County of Fulton

Mr. Virgil Ehle
Morrell Vrooman Engineers
Consultant Civil Engineers
Gloversville, New York

Dear Sir:

The application, plans and specifications filed by you with this Department in accordance with the provisions of Section 948 of the Conservation Law for the owner City of Johnstown, City Hall, Johnstown, New York, for raising the crest of the existing spillway are satisfactory to us.

The details of the proposed work are approved to the extent of the authority of the Superintendent of Public Works under the aforesaid statute.

This dam bears our new designation of #1720-3191 of the Mohawk River Watershed.

One set of formally approved plans and specifications is being returned for the owner's records.

Very truly yours,

M. A. Fobes
Deputy Chief Engineer

By: ____________________
A. Dickinson
Assoc. Civil Engineer

JRF:fs
Encl.
CC: Mr. F. W. Montanari

MORRELL VROOMAN ENGINEERS
CONSULTING CIVIL ENGINEERS
GLOVERSVILLE, N. Y.

April 18, 1963

New York State Department of Public Works
Division of Construction
Albany 1, New York

Attention: Mr. Albert Dickinson,
Assoc. Civil Engineer

Gentlemen:

Re: Water System Improvements
Dam #427 - Proposed Flashboards
City of Johnstown
Fulton County, New York

We have your letter of April 15, 1963 concerning the above project. We are having difficulty checking the discharge of the spillway as noted in paragraph 3 of your letter and we will appreciate it if we can resolve this question.

In our interview with your Mr. John Peck on January 4, 1960 regarding the raising of this dam, the requirements established were:

Run-off - 250 to 300 cubic feet per second per square mile
Minimum freeboard - 12 inches.

We agree with these standards.

We used a watershed of 3 square miles and a run-off of 300 cubic feet per second. The discharge capacity of the spillway was computed by Bazin's formula. The result is quite different from the capacity you note. We would appreciate receiving your comments.

Thank you for the plans you forwarded. It would seem well if we can have the partially filled out application in order to avoid confusion.

Very truly yours,
MORRELL VROOMAN ENGINEERS

By Virgil Ehle
Virgil Ehle

VE:ef

CORK CENTER CREEK RESERVOIR
CITY OF JOHNSTOWN #1732-311

El. top of Embankment = 1059.0' Elev. of Spillway Crest = 1054.0
Less 1'-0" of Freeboard = - 1.0' Add 1'-6" for Flashboard = + 1.5
Assumed Elev. of Max. Water = 1058.0 Elev. of Top of Flashboards = 1055.5

Elev. of Max. Water = 1058.0 Elev. of Max. Water = 1058.0
Elev. Top of Flashboards = 1055.5 El. base of Weir = - 1047.0
Depth of head = "H" = 2.5' Depth of Water = "d" = 11.0

Refer to Kings "Handbook of Hydraulics" 1954 edition pages 4-4 & 4-

Bazin Formula: $Q = \frac{2}{3}\sqrt{2g}\, L H^{3/2}\left(0.6075 + \frac{0.01476}{H}\right)\left(1 + 0.55\frac{H^2}{d^2}\right)$

$L = 60'0" - 2'0" = 58'0"$

$\sqrt{2g} = 8.02$

$\therefore Q = 0.667 \times 8.02 \times 58.0 \times 2.5^{3/2}\left(0.6075 + \frac{0.01476}{2.5}\right)\left[1 + 0.55\left(\frac{2.5}{11.0}\right)^2\right]$

$Q = \overbrace{0.667 \times 8.02 \times 58.0 \times 3.953}^{1225}(0.6075 + 0.0059)(1 + 0.0284)$

$Q = 1225 \times 0.6134 \times 1.0284$

$Q = 770$ cfs

Formula used was: Robert E. Horton's Formula for Broad Crested Weirs King's page 5-5

$Q = CLH^{3/2}$

$Q = 3.5 \times 58.0 \times (2.5)^{3/2}$

$Q = 804$ c.f.s.

Coefficient "C" of 3.5 is used because of the smoother surfaces of the concrete sections.

April 15, 1963

RE: Proposed Flashboards
Dam #427
Town of Johnstown
County of Fulton

Morrell Vrooman, Engineers
21 North Main Street
Gloversville, New York

Gentlemen:

The type of flashboards shown on the plan submitted with the application in reference to the above dam is not deemed satisfactory.

The large volume of water that would be suddenly released by automatically controlled flashboards added to flood stage runoff waters could possibly cause considerable damage to installations downstream from the dam. Due to this potential hazard approval cannot be given for construction of this type.

Our investigation shows that the crest of the spillway can be raised 18 inches and still be capable of discharging about 2-1/2 times the anticipated runoff. On this basis, a permanent type of construction; such as raising the crest with reinforced concrete, securely anchoring permanent flashboards, or installation of roller or radial gates to continuously control the water level; would be acceptable.

In the event it is decided to obtain the additional storage capacity as outlined above please submit three sets of prints of the revised plan for our approval.

Your tracing and a photostat copy of Sheet 8 of the intake details are enclosed.

Very truly yours,

E. W. Dayton
Deputy Chief Engineer

BY: ____________________
A. Dickinson
Assoc. Civil Engineer

AD/em
Encl.

MORRELL VROOMAN ENGINEERS
CONSULTING CIVIL ENGINEERS
GLOVERSVILLE, N. Y.

October 10, 1963

New York State Department of Public Works
State Campus Site, Washington Avenue
Albany 1, New York

Attention: Mr. Albert Dickinson

Gentlemen:

Re: Water Supply Improvements
Raising Cork Center
Storage Reservoir Dam
City of Johnstown, New York

For your review and approval we are transmitting herewith Application Form E-61A1 (2/62) together with three sets of plans and specifications as follows:

Specifications - Water Supply Improvements
Contract No. 2--Raising
Cork Center Dam

Plans - Water Works Improvements, Raising
Cork Center Storage Reservoir Dam,
as prepared by us and dated
September 1963.

We look forward to receiving your early approval for the referenced project.

Very truly yours,

MORRELL VROOMAN ENGINEERS

By Virgil Ehle
Virgil Ehle

VE:EF
Enc.

Johnstown water supply
Cork Center Reservoir
dam is on Keck Center Creek Flowing into Cayadutta Ck.
5 miles northwest of Johnstown. Reconstructed 1963: RAISED dam 15"
Drainage around lake is 2.6 sq miles
40 year peak rate of runoff is 300 cu. ft./sec.
spillway designed to discharge 900 cu. ft./sec
maximum height of dam is 40 FT
Designed maximum highwater crest above spillway is 2'-9"
with freeboard of 1'.
Concrete abutment
right bank gravel with cobble & boulders
LEFT " " " " " "
41 acres of water 128,000,000 gallons of water at normal elev
145,000,000 " " " at spillway elev
drainage by a 24" cast iron pipe
dam buit 1919

Form E-61A1 (2/62)

STATE OF NEW YORK

O.K. J.E. [illegible]

DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION
BUREAU OF WATERWAYS
ALBANY

Received October 14, 1963 Dam No. Orig #427 ~~427~~ 172C-3191

Disposition Design Approved October 17, 1963 Watershed Mohawk River

Foundation inspected

Structure inspected

Application for the Construction or Reconstruction of a Dam

Application is hereby made to the Superintendent of Public Works, Albany, N. Y., in compliance with the provisions of Section 948 of the Conservation Law (Chapter 602, Laws of 1959) for the approval of specifications and detailed drawings, marked Raising Cork Center Storage Reservoir Dam - Sheet 1

herewith submitted for the { ~~construction~~ reconstruction } of a dam herein described. All provisions of law will be complied with in the erection of the proposed dam. It is intended to complete the work covered by the application about November 22, 1963 (Date)

1. The dam will be on Keck Center Creek flowing into Cayadutta Creek in the town of Johnstown County of Fulton and 5 miles northwest of the City of Johnstown (Give exact distance and direction from a well-known bridge, dam, village, main cross-roads or mouth of a stream)

2. Location of dam is shown on the attached map or overlay of the Peck Lake quadrangle of the United States Geological Survey at latitude 43° 02' 14" N longitude 74° 27' 55" W

3. The name of the owner is City of Johnstown

4. The address of the owner is City Hall, Johnstown, New York

5. The impounded water will be used for water supply

6. Will any part of the dam be built upon or its pond flood any State lands? No

7. Does Section 179 of the Conservation Law (see page five of this form) apply to the above named stream? Yes........; No X If answer is yes, give Conservation Department's assigned number for permit to change or modify the stream

8. The area draining into the proposed pond or lake is .... acres; 2.6 square miles.

9. The computed 40 year peak rate of runoff used in the design is 300 cu. ft. per sec.

State criterion of method used in determining the peak rate of runoff N.Y.S. Dept. of Public Works - used and not exceeded on similar watersheds - also observed depth on this structure

10. The maximum height of the proposed dam above the bed of the stream will be 40 feet .... inches.

11. The designed maximum high water elevation above the spillcrest is computed to be 2 feet 9 inches; the designed freeboard as measured from the maximum high water elevation to the top of the proposed dam will be 1 feet 0 inches.

12. The open spillway of the ~~proposed~~ existing dam that will control the designed flood flow will be of concrete The width of the control section of

(State type, such as: vegetated earth, concrete, masonry, timber, rock filled crib, etc.)

the spillway, measured normal to the flow of water at the crest, will be 58 feet .... inches in the clear; facing down stream, the waters will be held at the right end by a concrete abutment the top of which will be 3 feet 9 inches above the spillcrest, and have a top width of 10 feet 0 inches; and at the left end by a concrete abutment the top of which will be 3 feet 9 inches above the spillcrest and have a top width of 10 feet 0 inches. The slope of the sides of the spillway will be vertical on .... (left) vertical on .... (right).

13. The spillway is designed to safely discharge 900 cu. ft. per sec.

14. The surface area of the proposed pond or lake will be 41 acres at the normal water elevation and 41 acres at the spillcrest elevation; the volume of the water impounded in the pond or lake will be 128,000,000 gallons at the normal water elevation and 145,000,000 gallons at the spillcrest elevation.

15a. The normal water elevation of the proposed pond or lake will be varying feet .... inches below the spillway crest, and will be maintained by means of a --; the pond or lake will be drained by means of a 24" cast iron pipe; provision will be made for supplying water to riparian owners downstream, during dry seasons, by means of dam in place since 1919.

15b. In addition to normal water control, provision must be made for a bottom draw-off if the pond is on a trout stream of constant flow. The draw-off will be by means of a --, designed to maintain an outflow of one-half of the minimum inflow of the stream of -- cu. ft. per sec. up to a maximum outflow of one cu. ft. per sec.

16. The maximum discharge through the spillway that controls the normal water elevation will be 900 cu. ft. per sec. during maximum high water.

17. If flashboards are to be used to control flood flow they must be of the automatic or self-tilting type, designed to fail or otherwise permit full discharge through the spillway when the flood waters reach a height of ...-... feet ...-... inches above the spillcrest.

18. If an overfall structure is used as a spillway, it shall be provided with an apron constructed of ...in place...; the thickness of the .... will be ...... feet ...... inches, the width ...... feet ...... inches across the stream and the length ...... feet ...... inches parallel to the stream.

19. Facing downstream, what is the nature of material composing the right bank? ....

Gravel with cobble and boulders

20. Facing downstream, what is the nature of the material composing the left bank? ....

Gravel with cobble and boulders

21. The natural material of the bed on which the proposed dam will rest is (clay, sand, gravel, boulders, granite, shale, slate, limestone, etc.) in place - no record

22. Are there any porous seams or fissures beneath the foundation of the proposed dam? ....

No appreciable leakage

23. State the character of the bed and the banks in respect to the hardness, perviousness, water bearing, effect of exposure to air and to water, uniformity, etc. Bed - boulders

Banks - gravel with cobble and boulders

24. Was the above soil information obtained from soil borings? ...-...; test pits? ...-...

25. State how much above the spillcrest elevation is the lowest part of the immediate upstream adjoining property or properties, ...4... feet ...0... inches.

26. Does this proposed pond or lake constitute any part of a public water supply? ...Yes...

27. State if any damage to life or to any buildings, roads or other property could be caused by any possible failure of the proposed dam Small intake reservoir - remainder farmland

28. The design, plans and specifications have been prepared under the supervision of: (Sign on applicable line below). MORRELL VROOMAN ENGINEERS

(a) By Morrell Vrooman Jr. .... P. E. License No. 22134
(Signature) Morrell Vrooman, Jr.

Address 21 North Main Street, Gloversville, New York

(b) .... U. S. D. A. Soil Conservation Service
(Signature) (Title: Engineer or Conservationist)

(c) .... N. Y. S. Conservation Department Engineer
(Signature) (Title)

(d) .... Other qualified engineer.
(Signature) (Title)

29. The raising of dam will be under the supervision of: (Sign on applicable line below).
(State which: Erection, Reconstruction or Repair)

CITY OF JOHNSTOWN

(a) By William H. McGregor P. E. License No. 27760
William H. McGregor, City Engineer

Address City Hall, Johnstown, New York (Telephone 6-9414

(b) .... U. S. D. A. Soil Conservation Service
(Signature) (Title: Engineer or Conservationist)

(c) .... N. Y. S. Conservation Department Engineer
(Signature) (Title)

(d) .... Other qualified engineer.
(Signature) (Title)

The foregoing information is correct to the best of my knowledge and belief, and the construction will be carried out in accordance with the approved plans and specifications.

CITY OF JOHNSTOWN, NEW YORK, Owner

By Mario H. Costa, authorized agent of owner.
Mario H. Costa, Mayor

Address of signer City Hall, Johnstown, New York Date October 7, 1963

INSTRUCTIONS

Read carefully, the law setting forth the requirements to be complied with in order to construct or reconstruct a dam.

Determine first whether the stream, across which the dam is to be erected or from which water for the proposed pond or lake is to be diverted, is under the jurisdiction of the Conservation Department. This information may be obtained upon request from the manager of the District Fisheries Office of the Conservation Department which has jurisdiction in the County where the stream is located, the Conservation Department, Bureau of Fish, State Campus Site, Albany 1, New York or the New York State Department of Public Works, Bureau of Waterways, Albany 1, New York.

Before a dam may be erected across a natural water-course, the riparian rights of other land owners (both upstream and downstream) must be considered and customarily their consent be obtained as such rights have been adjudged by the civil courts to be inalienable and inviolate.

The elevation of the impounded water should be maintained at a suitable level below the lowest contour of the adjoining properties thereby preventing inundation of the properties during the highest stage of the waters.

Each application for the construction or reconstruction of a dam must be made on this standard form, copies of which will be furnished upon request to the New York State Department of Public Works, Bureau of Waterways, Albany 1, New York. The application, properly executed, must be accompanied by three sets of plans and specifications. The plans must contain the following information:

a. A topographical plan (with contours) of the impounded area drawn to a suitable scale.

b. A profile and transverse section of the impounded area showing the proposed excavation, the normal water and possible high water elevations. A 1'–0" minimum of freeboard is to be provided between the top of the dam and the possible high water.

c. A longitudinal elevation and transverse section of the dam with all the necessary details of the related appurtenances, spillways, drains, etc.

d. A log of the soil information. Samples of the materials to be used in the dam and of the material upon which the dam is to be founded may be asked for, but need not be furnished unless requested.

No work of construction, reconstruction or repairs of the structure or structures shall be started until after the plans and specifications have been formally approved by the New York State Department of Public Works.

If the dam constitutes a part of a public water supply, application should also be made to the Water Resources Commission under Article V of the Conservation Law, as amended.

An application for the construction or reconstruction of a dam must be signed by the prospective owner of the dam or his duly authorized agent. The address of the signer and the date must be given as provided for in this application form.

DWR DAM INSPECTION REPORT

RB	CTY	YR AP.	DAM NO.	INS. DATE	USE	TYPE
33	18	63	3191	102369	03	2

AS BUILT INSPECTION

[/] Location of Sp'way and outlet

[/] Elevations

[/] Size of Sp'way and Outlet

[/] Geometry of Non-overflow section

GENERAL CONDITION OF NON-OVERFLOW SECTION [/]

[/] Settlement	[/] Cracks	[/] Deflections
[/] Joints	[/] Surface of Concrete	[/] Leakage
[/] Undermining	[/] Settlement of Embankment	[/] Crest of Dam
[/] Downstream Slope	[/] Upstream Slope	[/] Toe of Slope

no apparent repairs needed or minor repair that can be covered by routine maintenance

GENERAL COND. OF SP'WAY AND OUTLET WORKS /

[4] Auxiliary Spillway	[/] Service or Concrete Sp'way	[/] Stilling Basin
[/] Joints	[/] Surface of Concrete	[/] Spillway Toe
[/] Mechanical Equipment	[4] Plunge Pool	[/] Drain

[/] Maintenance

[/] Hazard Class

[3] Evaluation

C [/] Inspector

COMMENTS:

PART I – INVENTORY OF DAMS IN THE UNITED STATES
(PURSUANT TO PUBLIC LAW 92-367)

See reverse side for instructions.

FORM APPROVED
OMB NO. 49-S-72901

Reports Control Symbol
DAEN-CWE-17

IDENTIFICATION	(2) DIVISION	(3) STATE	(4) COUNTY	(5) CONGR DIST	(6) STATE	(7) COUNTY	(8) CONGR DIST	(9) NAME	(10) LATITUDE (North)	(11) LONGITUDE (West)	(12) REPORT DATE DAY MO YR
	NAD	NY	03	31				[illegible] RESERVOIR DAM	[illegible]	[illegible]	[illegible]

IDENTIFICATION (Continued)	(13) POPULAR NAME	(14) NAME OF IMPOUNDMENT
		[illegible] CREEK RESERVOIR

LOCATION	(15) REGION	(16) BASIN	(17) RIVER OR STREAM	(18) NEAREST DOWNSTREAM CITY - TOWN - VILLAGE	(19) DIST FROM DAM (mi)	(20) POPULATION
	[illegible]	[illegible]	[illegible] CREEK			

STATISTICS	(21) TYPE OF DAM	(22) YEAR COMPLETED	(23) PURPOSES	(24) STRUCTURAL HEIGHT (ft)	(25) HYDRAULIC HEIGHT (ft)	(26) IMPOUNDING CAPACITIES MAXIMUM (acre-ft)	(27) IMPOUNDING CAPACITIES NORMAL (acre-ft)	BLANK
	[illegible]	[illegible]		40	36	144	393	

REMARKS	(28) REMARKS
	[illegible]

PART II – INVENTORY OF DAMS IN THE UNITED STATES
(PURSUANT TO PUBLIC LAW 92-367)

See reverse side for instructions.

Form Approved
OMB No. [illegible]

Reports Control Symbol
DAEN-CWE-17

STATISTICS

(30) Crest Length (ft)	550
(31) Spillway Type	4
(32) Spillway Width (ft)	66
(33) Spillway Maximum Discharge (cfs)	706
(34) Volume of Dam (CY)	25555
(35) Power Capacity Installed (MW)	
(36) Power Capacity Proposed (MW)	
(37) No.	
(38) Navigation Locks Length (ft)	
(39) Width (ft)	
(40) Length (ft)	
(41) Width (ft)	
(42) Length (ft)	
(43) Width (ft)	
(44) Length (ft)	
(45) Width (ft)	

MISC DATA

Owner	Engineering By	Construction By
CITY OF JOHNSTOWN		

MISC. DATA (Continued)

Regulatory Agency

Design	Construction	Operation	Maintenance
DEC	DEC	DEC	DEC

MISC. DATA (Continued)

Inspection By	Inspection Date (Day / Mo / Yr)	Authority for Inspection
DEC	23 OCT 7[illegible]	ENCON LAW SECT 15-0507

REMARKS

Remarks
[illegible]

Mohawk

JAMES P. WELLS
CONSULTING ENGINEER
CUTLER BLDG. ROCHESTER, N.Y.

May 20, 1919.

Mr. A. H. Perkins,

Conservation Commission,

Albany, N. Y.

Dear Mr. Perkins:-

I am having sent from Johnstown the three blocks of concrete for test in regard to which I wrote you.

These blocks were made in the following manner: The proportionsof material were 1 part cement to three (3) parts of a mixture of bank sand and crusher dust and six (6) parts of crushed stone. They were made by taking a small amount at a time from several batches of concrete as it came from the mixer. They were made last June. The concrete was allowed to set and the cubes were then put in water for a week and since that time they have been out in the open and the forms have never been taken off.

Sincerely yours,

J. P. Wells

JPW/C

172C-3191
Mohawk

B. C. Form 33

FRANK M. WILLIAMS, STATE ENGINEER

D. B. LADU, SPECIAL DEPUTY STATE ENGR.

STATE OF NEW YORK
DEPARTMENT OF STATE ENGINEER AND SURVEYOR
ERIE CANAL, RESIDENCY NO.

SUBJECT:

June 10, 1919.

Conservation Commission,
Albany, N. Y. Attention of Mr. McKim.

Dear Sir:-

We have received three concrete cubes which bore one your tags, No. 259, and marked as "From Johnstown Water Works, Johnstown, N. Y., For dam at Cork Center on the Creek".

In accordance with your instruction we have tested these cubes for compressive strength. The results of the tests are as follows:-

Cube No.	Dimensions inches	Area sq.in.	Compressive Strength Total	Per Sq.in.
1	6.0x6.0x6.0	36.0	113,000 lbs	3,140 lbs
2	6.1x5.9x6.0	36.0	109,500 "	3,040 "
3	6.0x6.0x6.0	36.0	121,000 "	3,360 "
		Average		3,180 lbs

Yours very truly,

Russell S. Greenman

Sen. Asst. Engineer,
in charge of Tests.

MEMORANDUM REGARDING PROPOSED DAM #427 MOHAWK
ON CORK CENTER CREEK

CITY OF JOHNSTOWN, APPLICANT

Serial No.275

- -

The Papers relating to this dam, as received by the writer from Inspector McKim, were as follows:

(1) Application form IW56, dated July 9, 1917, completed in duplicate, and executed by Louis K. Maylender as President of the Board of Water Commissioners of the City of Johnstown.

(2) Engineer's Report (3 sheets in backer) signed jointly by W. E. Natanson, City Engineer, and James P. Wells, Consulting Engineer.

(3) Specifications in duplicate (13 sheets in backer)

(4) Certified copy of resolution adopted July 9, 1917, by the Board of Water Commissioners approving the plans, ordering the construction of the dam, etc. (4 sheets in backer)

(5) Pamphlet - 39th Annual Report of Board of Water Commissioners, City of Johnstown, N. Y., dated Dec. 31, 1916.

(6) General location and watershed map, marked (1) on U.S.G.S. Gloversville quadrangle.

(7) Portfolio containing blue prints of plans submitted in duplicate, (each containing 7 sheets size 24" x 36" numbered consecutively) showing plans, sections and details descriptive of the dam, spillway channel, reservoir and structures appurtenant thereto.

The following described data is lacking, although required by our printed instructions to applicants:

Engineer's explanation of stability of the overfall section, giving methods of computation and results as to -

(a) Overturning
(b) Sliding
(d) * * * Sufficiency of washwall.

The plans or specifications are lacking in clearness in the following particulars:

(1) Spillway channel bed. Paragraph 20 of specifications reads in part - "To break the flow of water, rubble-stone will be laid in the bed of the spillway channel so that about half the stone is imbedded in the concrete. All such stone shall be about 2 cubic feet in size and will be spaced about

three (3) feet apart center to center." Neither the plan, profile nor sections of such spillway channel, appearing on the drawings numbered 4, 5 and 6 (Acc. 8059, 8060, 8113) indicate in what portion such projecting stones are to be laid.

(2) Apron slope of the overfall section of the dam is neither marked on the plans nor were dimensions discovered from which it might be computed.

The site of the dam, as indicated on the U. S. G. S. map attached to the original copy of this memorandum, is in the Town of Johnstown, Fulton County, on Cork Center Creek, a branch of Cayadutta Creek, at a distance, measured along the stream, of about 4-1/2 miles above Sammonsville, and about 8-1/2 miles above the Village of Fonda. The U. S. G. S. sheets show the stream as flowing, throughout its entire length, in a very narrow valley, and the drawings submitted by the applicant indicate a total fall of about 750 feet between the water surface elevation at the proposed dam and that at the Village of Fonda (population 1,125). Immediately above Fonda the stream bed appears to have a fall of about 50 feet to the mile. At a point about 1-3/4 miles below the site of the proposed dam (storage capacity 22,000,000 cu.ft.) the U. S. G. S. sheet indicates a concentrated fall of about 350 feet in 1-1/2 miles, and Sammonsville lies almost immediately below.

The drainage area above the proposed dam site on Cork Center Creek is stated in the application as 2.86 square miles. The slopes are generally rolling or fairly steep throughout, and tributary stream beds short and almost straight. There is only one other small pond shown within the area and the proposed reservoir (water surface 50 acres) only appears to have a storage capacity above the spillway crest of about 2,400 acre inches, or 1.31" on the watershed.

The maximum probable flood from the entire watershed was estimated by the applicant's engineers as 572 cubic feet per second, although they further stated that the spillway had been designed to take care of 560 cubic feet per second per squaremile, or a total of 1600 cubic feet per second. The formula for maximum discharge, as developed by our Mr. McKim, indicates a probable flood of about 1315 cubic feet per second from the watershed under consideration while this Commission's enveloping curve (Acc. C-1618) indicates a probable flood discharge of about 270 cubic feet per second per square mile, or a total of about 770 cubic feet per second.

SPILLWAY CHANNEL CAPACITY

With stones set in its bottom, projecting upward about one foot and spaced about three feet center to center, as provided for by paragraph 20 of the specifications, and neglecting the retarding effect of the two curves in the channel, it does not appear that the section 20 feet wide, with a 5/10% grade, would discharge more than about 785 cubic feet per second when flowing full to the top, which is equivalent to a clear depth of about 5 ft. above the tops of such stones. This value has been determined by the use of the Chezy and Kutter formulas (assuming a value for the coefficient of roughness of .03 as recommended by R. E. Horton for cement channels with "dry-rubble surface", in good condition) and represents an average velocity of about 7.85 feet per second and only about 49%

of the discharge for which the overfall section is said to have been designed.

With a smooth cement bottom and neglecting the retarding effect of the two curves in the channel, it appears that the section 20 feet wide, with a 5/10% grade, would discharge the quantity for which the overfall section is said to have been designed, with a depth of about 4.9 feet, such value having been determined by the same method as stated in the last paragraph, except that the value of .014 for such channels in good condition was assumed for the coefficient of roughness.

Three curves are shown in the proposed channel, each of which changes the direction of flow by an angle nearly ninety degrees. At the first curve, which occurs immediately at the foot of the overfall section, the designer increases the height of the extreme side of the channel by about 33-1/3% to prevent overflow. At the second curve, which occurs at the middle of the section shown as 20 ft. wide (and for which the discharging capacities were computed as stated in the previous paragraph) both sides of the channel are indicated as the same height of 6 ft. (Section - Sta. 2+50, Sheet 6, Acc. 8113). It is possible that the channel might overflow on the outside of the curve at this point, although the drawings (Sheet 2, Acc. 8057) do not seem to indicate that a failure of such channel would impair the earth embankment to any great extent.

STABILITY OF THE OVERFALL SECTION

The section above the natural surface, as indicated by the elevation of 1047', on drawing 4 (Acc. 8059), was first investigated in accordance with the following assumptions:

(1) That the apron slopes 5.2 feet backward in a rise of 7 ft.

(2) That an open joint may form along a horizontal plane at the elevation stated.

(3) That the intensity of hydrostatic pressure upon the base of the section above such a joint might range from maximum at the back to zero at the intersection with the apron slope.

(4) That two-thirds of the area of the triangle representing the hydrostatic pressure upon the base of such a section would approximately represent the total of such pressure.

From an investigation of the forces acting upon the section formed as described above, it was found that the point of application of the resultant fell 13.6% outside the middle third of the base (using the length of that portion as a unit) when overtopped to a depth of four feet. The corresponding coefficient of stability was about 1.1, and the required coefficient of friction to prevent sliding was about 94%. If a coefficient of friction between

concrete surfaces of 70% may be safely assumed, then the point of application of the resultant of the forces acting would seem to fall well within the middle third when the total static pressure amounts to about 590 lbs., or approximately 33-1/3% of full uplift determined as described above. If a coefficient of friction of only 60% is assumed as safe between such surfaces, then the allowable static pressure would be about 195 lbs., or approximately 11% of full uplift determined as described above.

The complete section, as indicated on drawing sheet 4 (Acc. 8059) was first investigated with the following assumptions:

(1) That the apron slope is to be the same as already stated.

(2) That an open joint might form between the horizontal slab and the vertical upstream face of the overfall section.

(3) That the line of seepage follows entirely around the outline of the submerged portion of the dam and cut-off wall (4 ft. thick).

(4) That the intensity of hydrostatic pressure ranges from maximum at the top of the horizontal slab (Elev. 1047) on the up stream side of the section, and declines progressively along such a seepage line, as described above, to zero at an assumed open joint so located that the horizontal length of the base of the section would be about 11.6 ft.

(5) That the total area of that portion of the triangle representing the hydrostatic pressure which would apply against the vertical back of the section, and two-thirds of the areas of those portions of the same triangle which would apply against the remainder of the base of the section, would approximately represent the total of such hydrostatic pressure below the top of the horizontal slab behind the section under consideration.

(6) That ice pressure may be safely neglected.

(7) That the resistance to sliding offered by the paving slab in front and cohesion between vertical sections of the dam may be neglected.

(8) That the soil, described by the applicant's engineers as sandy loam will exert additional pressure upon the back of the dam and the front of the cut-off wall (4 ft. thick), in accordance with the general theory, taking into consideration the loads superimposed.

(9) That the approximate weight of the sub-soil may be safely assumed as 100# per cubic foot and that its plane of repose slopes approximately 2 on 3.

From an investigation of the forces acting upon the section formed as described above, and overtopped to a depth of 4 ft., it was found that the point of application of the resultant fell well within the middle third of the length of the base (on the upstream side of the middle), the value of the coefficient of stability was about 1.5, and that of the coefficient of friction between the material of the dam and the sandy loam or hardpan sub-soil required to prevent sliding would be about 74%, which may not be warranted. It also appears, however, that an intensity of resistance of only 690 lbs. per square foot exerted by the sub-soil in front of the down stream side of the cut-off wall, would reduce the coefficient of friction to about 45%, and it seems quite possible that such an assumption could be safely made.

The maximum sub-foundation load seems to maintain when the elevation of the water surface in the reservoir lies below the base of the overfall section. The intensity of such pressure was determined to be about 1-1/4 tons per square foot, acting upon a damp sandy loam or hard-pan, as described by the applicant's engineers.

RETAINING WALLS

The retaining walls for protecting the earth embankment at the ends of the overfall section, as outlined on the profile along the center line of the dam (drawing sheet 3, Acc. 8058), seem to be of rather scant dimensions, ranging as they do from a 2 ft. top width to a maximum width of 4 ft. at a point about 8 ft. below the top. The exposed total height of such walls on the up stream side of the overfall section is about 12 ft. and they must resist some static pressure as well as earth pressure. The length of these sections having a 12 ft. exposed height is short, yet their location is such that should a flood occur immediately after a failure the up stream side of the embankment would be badly eroded and this might result in the undermining of the core wall and failure of the entire structure.

Respectfully submitted,

John M. [signature]

JUNIOR ENGINEER.

To Mr. A. H. Perkins,

Division Engineer.

July 24, 1917.

.2300 (10-2061)
Acc. 106
Form W96

STATE OF NEW YORK — CONSERVATION COMMISSION

Dm 4-27 M.

SUBJECT: City [illegible] Water [illegible] FILE NO. [illegible]

Storage Reservoir Dam (#27 Mohawk) Serial #275 ACC. NO. C-3677

SHEET 1

COMPUTER [illegible] July 11, 1917 CHECKED BY ______ 19__

MADE IN CONNECTION WITH Typewritten memorandum dated July 24, '17.

REFERENCE Original Application and accompanying papers as listed on back of [illegible] CONT'D FROM ACC.

Spillway Channel Capacity (Station 100 [illegible], Acc. [illegible])
(Neglecting effect of rubble bottom and curves.) (Channel width = 20 ft., Acc. [illegible])

Maximum (Neglecting Velocity of Approach) Grade = .5% or 1 ft in 200

Using Weir Formula — Maximum Depth = 6 ft.

$C = \frac{3.35}{S^{0.12}} = \frac{3.35}{200^{0.12}}$ — log 3.35 = 0.585 — log 200 = 2.301

$\log 200^{0.12} = 0.276$ — $\times 10 = 23.01$

$\log C = 0.309$ — $\times 2 = 4.602$

$= 2.04$ — $\times .12 = .276\,12$

$Q = CLH^{\frac{3}{2}} = 2.04 \times 20 \times 6^{\frac{3}{2}} = 600.0$ c.f.s. (40.8; 14.7)

(This is too high as for [illegible] for weir with level [illegible] See U.S.G.S. W.S.P. 200, P. [illegible] and XII.)

Note: This is much smaller than assumed by the designing engineer in his application and report.

Correct for maximum possible velocity of approach.

$Q = Av = 1600$ c.f.s. ∴ $v_m = \frac{Q}{A} = \frac{1600}{\cancel{6\times 20}\ 120} = 13.33$ ft. per sec. ([illegible])

$h = .0155\,v^2 = 2.759$ ft. ($\times 178 =$) — $H = 6$ ft.

$Q = 2.04\left(b - \frac{n}{10}H\right)\left[(H+h)^{\frac{3}{2}} - h^{\frac{3}{2}}\right] = 870$ c.f.s. (Francis Formula) (Using [illegible] for sloping apron on weir) (Probably too [illegible] note opposite value [illegible] Q above. [illegible])

Using coefficient for broad crested weir (U.S.G.S. W.S.P. 200, p. 121 and [illegible])

$Q = 2.64 \times 20 \times 21.32 = \underline{1,125}$ c.f.s. (For weir with [illegible] front and back. Discharge therefore [illegible] high [illegible] velocity too high.)

Note: Both of the last two computed discharges were based on the assumption that the spillway channel at Sta. 100 (Acc. 859-G) would flow full to top with correction for full velocity head required to pass the 1600 c.f.s. through such a water cross section. ~~Depth would probably be much less than six feet and it seems improbable that the resulting high velocity would [illegible] any such values of discharge~~ Channel bottom [illegible]

Using Chezy and Kutter Formulas $v = C\sqrt{rs}$

Assuming channel flowing full throughout entire length — $A = 6 \times 20 = 120$

$n = .03$ (Dry rubble) — $S = \frac{h}{l} = \frac{.5}{100} = .005$ — $r = \frac{}{p} = \frac{}{32}$

$C = 42$ (Trautwine, 568)

$v = 61.9\sqrt{3.75 \times .005} = 8.48$ ft. per sec.

$Q = Av_m = 120 \times 8.48 = 1,018$ c.f.s. See Shts. 2 and 15 also.

or — for $n = .027$ ... $Q = \underline{1,120}$ c.f.s.

CONT'D ON ACC. [illegible]

1-23-17-2500 (16-2901)
Acc. 106
Form W86

STATE OF NEW YORK—CONSERVATION COMMISSION Dm-42/M .

SUBJECT: City of Johnstown — FILE NO.: [illegible]
Dam 427 Mohawk (Serial #275) — ACC. NO. C-3678
See Sheet #1. — SHEET 2
COMPUTER: [illegible] July 12 1917 — CHECKED BY: ______ 19__
MADE IN CONNECTION WITH: ______
REFERENCE: ______ — CONT'D FROM ACC. C-3677

Spillway Channel Capacity continued from sheet 1.

Chezy Formula – Continued

If stones are not imbedded in bottom of spillway channel as seemingly required by ¶ 20 of specifications and bottom was finished smooth, then value of "n" of 0.014 might be warranted (Acc. 1730)

$v_m = c\sqrt{rs} = 131.4 \times .137 = 18.0$ ft/sec. (Ref. Sh[illegible])

$Q = 120^{\square} \times 18$ ft/sec $= 2160$ c.f.s. with channel flowing full to top which would probably not maintain.

Determining depth required to discharge 1600 c.f.s.

Using value of "n" for concrete lined channel in good condition $= 0.014$ (Acc. 1730)

Try depth of 5 ft in channel.

$v_m = c\sqrt{rs} = 129.95\sqrt{3.333 \times .005} = 16.7$ ft/sec. (.01668)

$r = \dfrac{A = 100^{\square}}{p = 30} = 3.33$

$Q = 100 \times 16.7 = 1,670$ c.f.s.

Try depth of 4.9' in channel

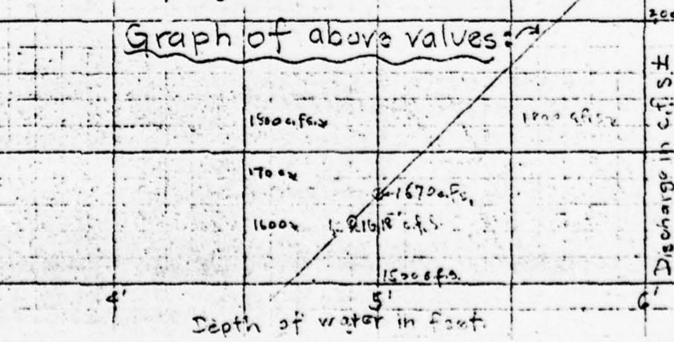
$r = \dfrac{A = 4.9 \times 20 = 98}{p = (2 \times 4.9) + 20 = 29.8} = 3.29$

$v_m = c\sqrt{rs} = 129.4\sqrt{3.29 \times .005} = 16.5$ ft/sec. (.1276, .01646)

$Q = 98 \times 16.5 = 1,618 \pm$ c.f.s.

Curves in channels will tend to reduce the probable discharge while steeper grade below will tend to increase it

Graph of above values:



Depth = 4'
$A = 80$
$r = \dfrac{}{p = 28} = 2.855$
$v = 126.9\sqrt{2.855 \times .005} = 15.17$
$Q = 80 \times 15.17 = 1,214 \pm$
For smooth channel.

CONT'D ON ACC. C-3679

STATE OF NEW YORK—CONSERVATION COMMISSION DM-427M

SUBJECT City of Johnstown FILE NO.

Dam 427 Mohawk (Serial #275) ACC. NO. C-3679

See sheet #1. SHEET 3

COMPUTER J. F. Henry July 11, 1917 CHECKED BY ______ 19__

MADE IN CONNECTION WITH

REFERENCE Cont'd from Acc. C-3678

Overfall Section – Discharging Capacity

Designing Engineer reports as follows (Ref. App. #275)

Watershed 2.86 Sq. M. (2.86 × 640 = 1,831 Acres)

Water surface 50 Acres. (43,560 × 50 × 7 = 15,246,000 c.ft) above natural surface

Reservoir capacity 22. M. C. F.

Spillway Length 60 ft ✓

May be overtopped 4 ft. {48" × 50 = 2400 Acre inches storage; 2400 ÷ 1831 = 1.31" on watershed.}

Resulting discharge 1600 c.f.s.

or 560 c.f.s./Sq. Mi.

Spillway divided into 3 sections by bridge supports.

Each end section = $\frac{\cancel{60}\,58 - 2}{3}$ = 19.333 ft. and Say ½ end contractions

Middle section 19.333 ft and Say 1½ " "

Breadths – Effective as corrected for end contractions

End sections: $b = 19.33' - \frac{0.75 \times 4}{10}$ (0.3) = 19.03 ft. Francis Form.

19.03

Middle Section: $b = 19.33 - \frac{.5 \times 4}{10}$ (.60) = 18.73 ft (" ")

Approximate effective length = 56.79 ft Total.

Discharge by W.S.P. 200, Tables 9 & 12, section m:

28.19 c.f.s × 1.11 × 56.79 ft = 1,778± c.f.s.

28.65± c.f.s. × 56.79 ft = 1,630± c.f.s. E.S.C. curve

1600 c.f.s. probably conservative

McKibbin's Formula for flood discharge:

$Q = 600 A^{\frac{3}{4}} = 600 \times 2.86^{\frac{3}{4}} = 1,314 \pm$ c.f.s. (2.86 ... 23.39)

Capacity at Spillway overfall section OK.

(Height in feet.)

Cont'd on Acc. C-3680

STATE OF NEW YORK—CONSERVATION COMMISSION Dm 427M

Subject: City of Johnstown

Dam 427 Mohawk (Serial # 275)

File No.

Acc. No. C-3680

Sheet 4

Computer: W Henry 12 Jy 1917 Checked by ________ 19__

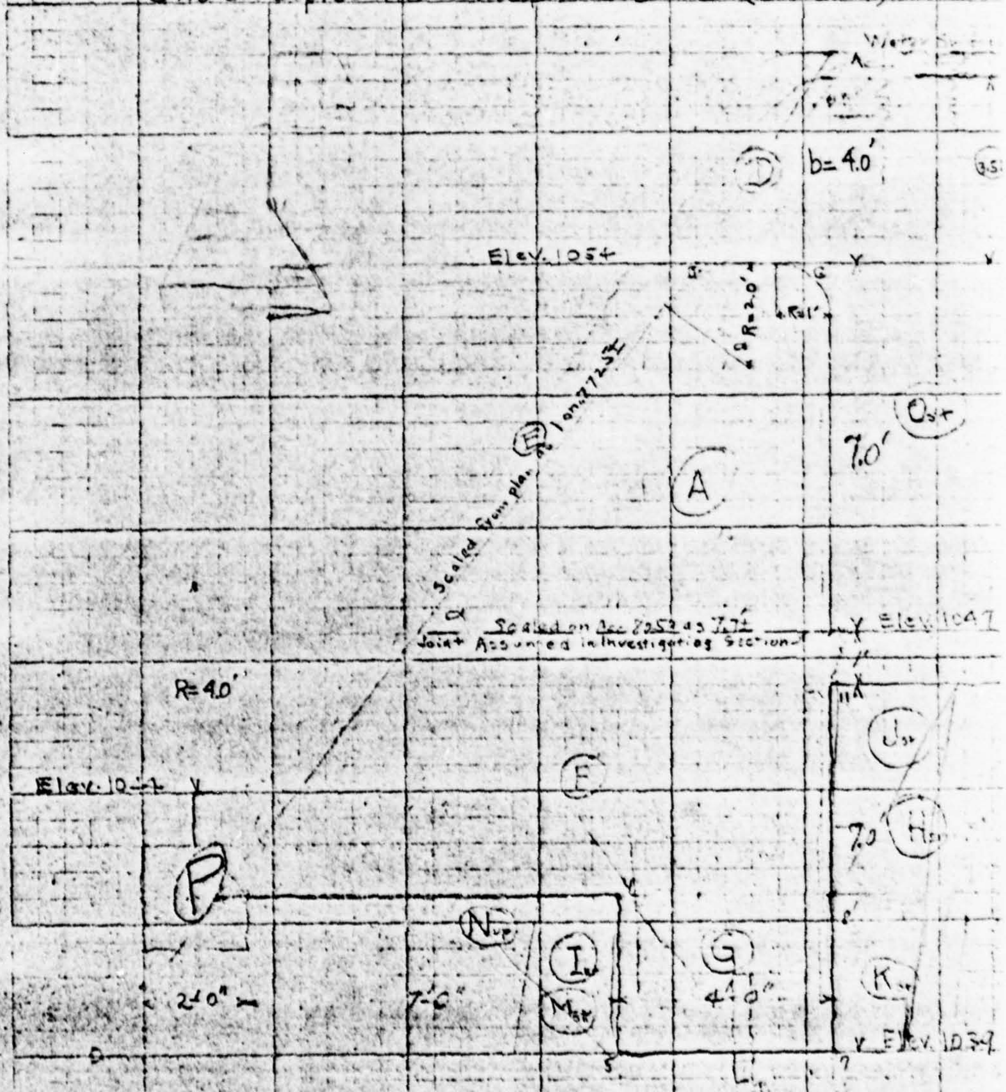
Made in Connection With

Reference

Comp's from Acc. C-3679

Overfall Section ~ Resistance against overturning and sliding

Sketch copied from Map Acc. 8059 (Sheet 4.)



STATE OF NEW YORK—CONSERVATION COMMISSION

SUBJECT: City of Johnstown — FILE NO. Dm 427M

Dam 427/Mohawk (Serial # 275) — ACC. NO. C-3681

SHEET 5

COMPUTER: J.W. Henry 12 Jy 1917 CHECKED BY

MADE IN CONNECTION WITH

REFERENCE — CONT'D FROM ACC. C-3680

Overfall Section Resistance (Cont'd.) Sketch on Sht 4

Neglecting Ice Pressure

Investigating section 1-ft. thick, overtopped 4 ft.

Assuming horizontal joint at intersection of natural surface line with back of overfall section (El. 10+7)

(A) Enveloping trapezoid

	Area	Arm	Moment
Area: $\frac{2.5\pm + 7.7\pm}{2} \times 7$ =	+35.7		
Arm: $= \frac{2l^2+2ll_o-l_o^2}{3(l+l_o)}$ =		4.9±	
Moment:			+176.0
(C) Portion cut from back of crest			
Area: $\frac{.785 \times 4}{4}$ =	−0.2		
Arm: (Approximately) 7.7−.2± =		7.5±	
Moment:			−1.5
(B) Portion cut from front of crest.			
Area: $\tan\alpha = \frac{1}{.7725} = 1.293\pm$; $\angle\alpha = 52°17'\pm$; $\frac{52.353°}{360°} = \frac{1.825\pm}{.7854 \times 16}$			
$\left(\frac{2.0' \times 1'}{2} \times 2\right) - 1.825$ =	−0.175		
Arm (Approximately) 7.7−2.4±=		5.3±	−0.9
Total Net Moment (Area × Ft.)			+173.6

M_{r} = " " " in Ft. Lbs. = 140# × 173.6 = 24,300±

CONT'D ON ACC.

STATE OF NEW YORK—CONSERVATION COMMISSION

Dm 427M

SUBJECT: City of Johnstown
Dam 427 Mohawk (Serial No. 275)
See Sheet #1.

FILE NO.: [illegible]
ACC. NO. C-3682
SHEET 6

COMPUTER: [illegible] July 13, 1917 CHECKED BY: ____ 19__

MADE IN CONNECTION WITH: ____

REFERENCE: ____ CONT'D FROM ACC. C-3681

Overfall Section: Investigation of resistance (Cont'd)

	Area	Arm	Moment
Water Resisting Moments:			
(B) Neglected.	—	—	—
(C) (From sht #5)			+ 1.5
(D) Prism above plane of crest;			
Area: (Assumed as equal to ½ the section of prism 2.5 × 4') =	+ 5.0		
Arm: $7.7 - \frac{2.5}{3}$ =		6.9'	
Moment =			+ 34.5
(E) Prism of water sliding on apron neglected.			
Total Net Moment (Area × Ft.) =			36.0
$M_{r(water)}$ = " " " in Ft. Lbs = $(62.5^{\#} \times 36.0)$ =			2,250

M_r = Total Resisting Moment = $M_{r(mas)} + M_{r(water)}$ = 26,550

$M_{o(static)} = \frac{\gamma (H-b)^2 (H+2b)}{6}$

$= \frac{62.5 (11-4)^2 (11+2\times4)}{6}$ = 9,700

$M_{o(uplift)} = \frac{\gamma H \times l}{2} \times \frac{2}{3} = \left(\frac{62.5 \times 11 \times 7.7}{2} \times \frac{2}{3} \times 7.7\right)$ = 9,060

M_o = Total Overturning Moment 18,760'#

$M_r - M_o$ = 7,790'#

Vertical Component of Resultant

	Areas (shts. 5 & 6)		Weight
Masonry			
(A)	+ 35.7		
(B+C)	− 0.39		
	35.3± Cu. Ft. Masonry at 140#/cu. ft. =		+ 4,950
Water			
(C)	+ 0.215 (sht 5)		
(D)	+ 5.0	5.215 × 62.5#/cu. ft. =	+ 326±
$+V_o$ = Total downward pressure			= + 5,276±

STATE OF NEW YORK — CONSERVATION COMMISSION Dm 4-27 M

SUBJECT City of Johnstown — FILE NO. [illegible]

Dam 427 Mohawk (Serial #275) — ACC. NO. C-3683

See Sheet #1 — SHEET 7

COMPUTER [illegible] 13 Jy 1917 CHECKED BY ______ 19__

MADE IN CONNECTION WITH ______

REFERENCE ______ CONT'D FROM ACC. C-3683

Overfall Section – Investigation of Stability Contd.

Vertical Component of Resultant Contd.

$+V_e$ (From sht. 6) $+5,276^{\#}$

Uplift (Sht. 6) $-1,766^{\#}$

$V_E =$ Net $= +3,510^{\#}$

$U = \frac{M_r - M_o}{V_E} = \frac{7,790'^{\#}}{3,510^{\#}} = 2.22'\pm$ {Arm of V_E, 13.6% outside middle third.

$\frac{l}{3} = \frac{7.7}{3} = 2.567\pm$ Length of middle third

Note: Since the point of application of the resultant falls outside the middle third, measured by the usual practice, the dam is said to be unsafe as the ~~concrete at the~~ concrete at the upstream portion of the section must resist tension which is not permitted by best practice. (See below and on sht. 8)

Sliding — Stability against

$H_o = \frac{\gamma H + \gamma b}{2} \times h = \left[\frac{62.5(11+4)}{2}\right] 7 = 3,282^{\#}\pm$

$F = \frac{V_E}{H_o} = \frac{3,510}{3,282} = 1.068\pm$ Coefficient of stability

$f = \frac{H_o}{V_E} = .936$ {Coefficient of friction should not exceed 70 or 75%

Seemingly unsafe against Sliding

Neglecting Uplift. (Sheets 6 & 7)

$U_1 = \frac{M_r - M_{o_1}}{V_{e_1}} = \frac{26,550'^{\#} - 9,700^{\#}}{5,276} = U = 3.195$ {Arm of V_e, 19.7% inside middle third.

$F_1 = \frac{V_{e_1}}{H_o} = \frac{5,276}{3,282} = 1.61\pm$ Coefficient of stability

$f_1 = \frac{H_o}{V_{e_1}} = .623\pm$ {Coefficient of friction seemingly O.K.

F.S. = 1

STATE OF NEW YORK—CONSERVATION COMMISSION Dm 427 M

SUBJECT City of Johnstown FILE NO.

Dam 427 Mohawk (Serial #275) ACC. NO. C-3684

See Sheet 7. SHEET 8

COMPUTER 19 CHECKED BY 19

MADE IN CONNECTION WITH

REFERENCE CONT'D FROM ACC. C-3682

Overfall Section - Investigation of Stability Contd.

Uplift Permissible in accordance with previous assumptions.

$$U_2 = \frac{l}{3} = \frac{Mr - (M_{o_1} + \frac{2}{3} \times 7.7 P_u)}{V_{E_1} - P_u} = \frac{2.567}{\frac{7.7}{3}} = \frac{16,850 - 5.13 P_u}{5,276 - P_u}$$

$$2.567 \times 5,276 - 2.567 P_u = 16,850 - 5.13 P_u \quad (13,530)$$

$$2.56 P_u = 3,320$$

(N.B. See revision below) $P_u = 1,275^{\#}$ or 73.6% of Total Uplift. As far as overturning is concerned (See Further Revision Below)

Sliding

$$F_2 = \frac{V_{E_1} - P_{u_2}}{H_o} = \frac{5,276 - 1,295}{3,282} = \frac{3,981}{3,282} = 1.21 \text{ Coefficient of Stability}$$

$$f_2 = .826$$ Coefficient of Friction Preferably should not exceed .60

$$f_3 = \frac{H_o}{V_{E_1} - P_{u_3}} = \frac{3,282}{5,276 - P_{u_3}} = .60$$ Recomputing allowable uplift for sliding with coefficient of friction of .60

$$.60 \times 5,276 - .60 P_{u_3} = 3,282 \quad (3,165.6)$$

$$P_{u_3} = \frac{3,282 - 3,165.6}{.60} = \frac{116.4}{.60} = 195^{\#}$$ or 11% of Total Uplift

$$f_4 = \frac{3,282}{5,276 - P_{u_4}} = .70$$

$$.70 \times 5,276 - .70 P_{u_4} = 3,282 - 3,693.2 = -411.2 \quad (3,693.2)$$

$$P_{u_4} = \frac{411.2}{.70} = 588$$ or 33.3% of total uplift

Unit Stresses with full upward pressure

$$P_a = \frac{V_E}{l} + \frac{6 V_E d}{l^2} = \frac{3,510}{7.7} + \frac{6 \times 3,510 \times (\frac{7.7}{2} \times 4.215)}{7.7 \times 7.7} = 456 + 3,033 \quad (1,060, 59.29) = 3,489^{\#}/\square'$$ Compression or 24.2 $^{\#}/\square''$

$$P_b = \frac{V_E}{l} - \frac{6 V_E d}{l^2} = 456 - 3,033 = 2,577^{\#}/\square'$$ Tension or 17.8 $^{\#}/\square''$ [illegible]

O.K.

Using class B (1:2:5) as specified, according to Phil. Pa. Municipal testing laboratory experiments factor of safety for 2 Mo. = 126 and for 6 Mo. = 13.3

CONT'D ON ACC.

STATE OF NEW YORK—CONSERVATION COMMISSION Dm 427M

SUBJECT City of Johnstown FILE NO. [illegible]

Dam #27 Mohawk (Serial #275) ACC. NO. C-3685

See sheet #1 SHEET 9

COMPUTER [illegible] July 16, 1917 CHECKED BY 19

MADE IN CONNECTION WITH

REFERENCE Con'd from Acc. C-3[illegible]

Overfall Section Investigation of Stability Cont...

Investigation at horizontal plane at highest elevation of sub foundation below overfall section (See red crayon line on sketch sht #4) but assuming that 4ft cut off wall built monolithic with dam

Neglecting icepressure as before

Considering section 1ft. thick longitudinally, overtopped to depth of 4 ft.

Assuming: joint in paving slab behind dam and

Intensity of static pressure as maximum at intersection of top of paving slab with back of dam (Elev 1047) and declines gradually along a line of seepage following the outline of the section and becoming 0 at an assumed point at the point of intersection of the slope of the apron extended, and the horizontal plane at the base of the dam (See red lines on sketch sht. 4)

(Revised on sheet 23, assuming that cut off wall is 100% efficient.)

Resistance to sliding due to paving in front of dam has been neglected.

Assuming that the adjacent sand or loam soil will exert pressures on the back of the section and the front of the cut off wall (4ft. thick) in accordance with the results obtained by the usual formulas taking weight of the loam as 100#/cu.ft. and natural slope of same as 2 on 3

		Area	Arm	Moment	
(A)	4.9' + (11.6 - 7.7) 3.9	+35.7	8.8'		+31
(B)	5.3' + 3.9'	−0.175	9.2'		−
(C)	7.5' + 3.9'	−0.215	11.4'		−
	Net for section above El. 1047			= +310	

(F) Trapezoid below natural surface and above top of cut off wall. (See sketch on Sht. 4)

Area: 7.7' + 11.6' × 5' = +48.2

Arm: (2 × 11.6) + 7.7 ... = 6.72

Moment: +324.0

Masonry Resisting Moment Forward to Sht. 10 + 634.8

CONT'D ON ACC.

STATE OF NEW YORK—CONSERVATION COMMISSION

SUBJECT: City of Johnstown — FILE NO.: [illegible]

Dam 427 Mohawk (Serial # 275) — ACC. NO. C-3687

Sec. Sheet #1. — SHEET 11

COMPUTER: [illegible] Jy 16, 1917 CHECKED BY: ________ 19__

MADE IN CONNECTION WITH:

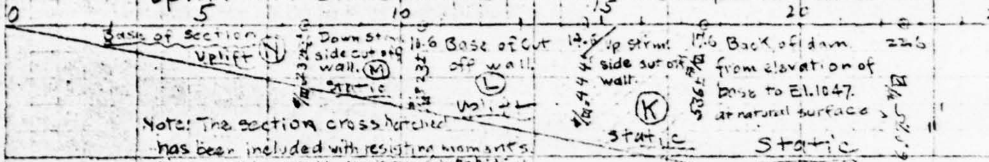
REFERENCE: — CONT'D FROM ACC. C-3686

Overfall Section, Investigation of Stability Continued

(Assumptions as stated on sht. 9.)

Mo = Overturning Moments

Uplift and Static above natural surface $\gamma H = 62.5 \times 11 = 687.5$



Note: The section cross hatched has been included with resisting moments. Since wall is probably thick enough to bear all the stresses coming upon it.

N.B. Excepting those portions of the above force diagram which act on the back of the dam and buttress wall, only 2/3 rds of the areas have been considered as effective. [illegible]

(J31) Rectangular portion (Total area used.)

			Moment
Force: $536 \times 5'$	= 2,680#		
Arm: $\frac{5.0}{2}$	=	2.5	
Moment:			6,700'#
Triangular portion (Total area used)			
Force: $\frac{687.5-536}{2} \times 5$ =	379#		
Arm: $\frac{2}{3} \times 5$ =		3.33'	
Moment:			1,263'#

(L32) Rectangle: ($\frac{2}{3}$ Area used)

Force: $323 \times 4 \times \frac{2}{3}$ =	861#		
Arm: $\frac{4}{2} + 7.6$	=	9.6'	
Moment:			8,260'#
Triangle: (2/3 Area used.)			
Force: $\frac{\cancel{354}-323}{2} \times 4 \times \frac{2}{3}$ =	161.1		
Arm: $7.6 + (\frac{2}{3} \times 4)$	=	10.27'	
Moment:			1,654'#

Overturning moments forward to sheet 12 = 17,877'#

CONT'D ON ACC.

STATE OF NEW YORK—CONSERVATION COMMISSION Dm427M

SUBJECT City of [illegible] FILE NO. [illegible]

Dam 427 Mohawk (Serial #275) ACC. NO. C-3686

See sheet #1. SHEET 10

COMPUTER [illegible] July 10, 1917 CHECKED BY ________ 19__

MADE IN CONNECTION WITH

REFERENCE CONT'D FROM ACC. C-3685

Overfall Section — Investigation of Stability Continued

(Assumptions as stated on Sht. 9)	Area	Arm	Moment	Ft×F
Masonry resisting moment forward from Sht. 9			+634.0	
(G) Section of 4 ft. cut off wall:				
Area: 4'×3±' =	12±			
Arm: 11.6± − 2'		9.6		
Moment:			+115.1±	
Total net masonry resisting moment (Area × Ft) =			749.1	
$M_{r(mas)}$ = " " " " " 140#×749.1 =			104,874#	
$M_{r(water)}$ = (Sht. 6.) " water (Error carried)			+2,250±	
Moment arm too short but usually neglected anyway				
(H_{earth}) For resisting moment due to pressure of earth in back of dam, see computation on sht. 18-20 for insertion here. (See sheet #20)				
Frictional resistance = 830# (sht. 20)				
Arm:		11.6		
Moment			9,630	
(I_{earth}) (Omitting pressure superimposed by dam) ?				
Friction = 35×1289# = 45.1# (Ref. C-3583)				
Arm and Moment:		7.6'	343±	
$M_{r(earth)}$			9,973±	
Static Resisting Pressure:				
(K_{static}) Rectangular portion: ($\frac{2}{3}$ of areas used)				
Force: 444#/□' × 3½ × $\frac{2}{3}$ = 888				
Arm:		1.5'+		
Moment:			1,332	
Triangular Portion (2/3 area used)				
Force: $\frac{536-444}{2}$ × 3 × $\frac{2}{3}$ = 92±				
Arm: $\frac{3}{3}$		1.0'		
Moment:			92±	
$M_{r(static)}$			1,424	
M_r = Total of Resisting Moments			= 118,347	

CONT'D ON ACC.

22-17-2500 (10-2961)
Acc. 200
Form W80

STATE OF NEW YORK—CONSERVATION COMMISSION

Dm 427M

SUBJECT City of Johnstown
Dam 427 Mohawk (Serial # 275)
(See Sheet 1)

FILE NO. [illegible]

ACC. NO. C-3688

SHEET 12

COMPUTER C. W. Henry Jy 17 1917 CHECKED BY ______ 19__

MADE IN CONNECTION WITH ______

REFERENCE ______ CONT'D FROM ACC. C-3687

Overfall Section-Investigation of Stability Continued
Assumptions as stated on Sht. 9.

Overturning Moments cont'd.		Force	Arm	Moments 1#
	Total of overturning moments forward from Sht. 11 =			17,877
I Earth	Earththrust on cut off wall			
	Force: (Neglecting superimposed load which reduces moment arm.)	128.9 #		
	Arm: (Neglecting reduced length of moment arm which would result if superimposed load is considered.)		2.0	
	Moment:			258 1#
M st.	Rectangle: (2/3 area used)			
	Force: 232 x 3 x 2/3 =	464 #		
	Arm: 3/2 =		1.5	
	Moment: =			696 1#
	Triangle: (2/3 area used)			
	Force: (323 − 232)/2 x 3 x 2/3 =	91 #		
	Arm: 3/3 =		1.0	
	Mom =			91 1#
N up	Triangle (2/3 Area used)			
	Force: 232/2 x 7.6' x 2/3 =	588 #		
	Arm: 7.6 x 2/3 =		5.07	
	Moment =			3,000 #
H earth	(Shts. 1 2 3 2 0) 2,370 # x		1.28	3,033
O sh	Above channel bottom			
	Overturning moments forward to Sht. 13			= 24,955 1#

Cont'd on Acc.

STATE OF NEW YORK—CONSERVATION COMMISSION Dm 427 M

SUBJECT City of Johnstown FILE NO [illegible]

Dam 427 Mohawk (Serial #275) ACC. NO. C-3689

(See Sheet 1) SHEET 13

COMPUTER W Henry Jy 17 1917 CHECKED BY ____ 19__

MADE IN CONNECTION WITH

REFERENCE CONT'D FROM ACC. C 3689

Overfall Section Investigation of Stability Continued
(Assumptions as stated on Sht. 9)

Overturning moments continued

	Force	Arm	Moment
Overturning moments forward from sheet #12			24,955 #
(O5) Continued: (Sketch on Sht. 12)			
Rectangle			
Force: (4×62.5)×7 =	1,750 #		
Arm: 7/2 =		3.5'	
Moment			6,125'#
Triangle			
Force: (~~687.5~~ 437.5 ~~−250~~)/2 ×7 =	1,531 #		
Arm: 7/3 =		2.33'	
Mom.			3,565'#
Mo = Total of Overturning Moments			= 34,645'#
Mr−Mo = (Mr on Sht. 10.)			83,702'#

Vertical Components

Earth Friction (H) & (I) (Sht. 10.) [illegible]	+ 875 #		
Areas			
Masonry			
A-(B+C) 35.3 Cu.Ft. Sht. 6.			
(F) 48.2 " " (" 9.)			
(G) 12.0 " " (" 10)			
95.5 " " × 140 #/cu.ft. =	+13,380 #		
Water (Sht. 6)	+ 326 #		
Total downward pressure =		+14,581 #	
Uplift Forces			
(L) 861 # (Sht. 11)			
(Y) 588 # (Sht. 12)		−1,449 #	
Ve = Net			13,132 #

U = (Mr−Mo)/Ve = 6.37'±   L/3 = 11.6/3 = 3.87'±   Dif = 2.50' (64.6% inside middle third)

CONT'D ON ACC.

1-22-17-2500 (10-2061)
Acc. 100
Form W66

STATE OF NEW YORK—CONSERVATION COMMISSION DM 427 M

SUBJECT: City of Johnstown — FILE NO.
Dam 427 Mohawk (Serial #275) — ACC. NO. C-3690
(See Sheet 1) — SHEET 14
COMPUTER: [illegible] Jy 17, 1917 CHECKED BY ____ 19__
MADE IN CONNECTION WITH
REFERENCE — CONT'D FROM ACC. C-3690

Overflow Section — Investigation of Stability — Continued.
(Assumptions as stated on sht. 9.)

Sliding

$$F = \frac{V_E}{H_o} = \frac{13{,}132^{\#}}{9{,}006^{\#}} = 1.46$$ (Ref - Below.)

{Too high without using the resisting power or effect of given opposed load of earth in computing (I) on sheet 12 which would increase this value. Coefficient of Stability

$$f = \frac{H_o}{V_E} = \quad = .736$$

{Concrete on sandy loam should not exceed .35 to .45
Continued on sheet 20 and 22 and recomputed with partial uplift on sht 23

Subfoundation loads

Section overtopped to depth of 4'.

Maximum

$$P_a = \frac{V_E}{l}\left[1 + \frac{6\left(U - \frac{l}{2}\right)}{l}\right] = \frac{13{,}132}{11.6}\left[1 + \frac{6 \times .57}{11.6}\right] = 1{,}470^{\#} \text{ O.K.}$$

$$P_b = \quad = 1{,}132 \times [1 - .295] \qquad 800^{\#}$$

Reservoir Empty

Maximum

$$U_1 = \frac{M_{r_1} - M_{o_1}}{V_{E_1}} = \frac{111{,}382^{\#}}{14{,}255^{\#}} = 7.84$$

(Ref. below) (Ref. below)

Diff = 0.11; $\frac{2}{3}$ = 7.73

{2.8 % beyond middle third O.K. Moment arm measured upstream from point assumed as toe.

$$P_a = \frac{V_{E_1}}{l}\left[1 + \frac{6\left(U - \frac{l}{2}\right)}{l}\right] = \frac{14{,}255}{11.6}(1 + 1.051) = 2{,}520^{\#} \; ?$$

{Fairly high but probably OK

Horizontal Component of Resultant (Used above.)

	Rectangle	Triangle		
O_{st}	+1,750	+1,531#	(Sht. 12)	
J_{st}	+2,680#	+ 377	(" 11)	
F_{Er}	+2,370#		(" 12)	
K_{st}	+ 888#	− 92#	(" 10)	
I_{Er}				− 129#
M_{st}				− 464#
				− 91#
		+ 7,698#		
		+ 9,690#		
		− 684#		− 684#

H_o = Net Total = + 9,006 # as Used above

		M_{r_1}	M_{o_1}	V_{E_1}
M_r (Masonry)	(Sht. 10)	+1,04,700'#		13,380#
(H_{earth})	(" 10, 12)	+ 9,630'#	− 3,033'#	830#
(I_{earth})	(" 10, 12)	+ 343'#	− 258'#	45#
		+114,673'#		
		− 3,291'#		
$M_{r_1} - M_{o_1}$		= 111,382'#	V_{E_1}	= 14,255# (Used above)

CONT'D ON ACC.

STATE OF NEW YORK—CONSERVATION COMMISSION DM 427M

SUBJECT City of Johnstown FILE NO.

Dam 427 Mohawk (Serial #275) ACC. NO. C-3691

(See sheet #1.) SHEET 15

COMPUTER [illegible] Jy 17, 1917 CHECKED BY ______ 19__

MADE IN CONNECTION WITH

REFERENCE CONT'D FROM ACC. C-36[illegible]

Overfall Section–Investigation of Stability Continued

Assuming maximum probable flood equal to values computed from formula derived by Mr McKim ($Q = 600 A^{\frac{2}{3}}$) and recomputing stability of the section overtopped to the depth thus indicated.

Neglecting ice pressure

Assuming open joint at intersection of natural surface line with back of overfall section (Elev. 047)

Investigating section 1 ft thick overtopped to depth as computed below.

$Q = 600 \times (2.86)^{\frac{2}{3}} = 1,314$ c.f.s (Ref– Sht. 3.)

Effective suppressed weir length (Ref. Sht. 3.)

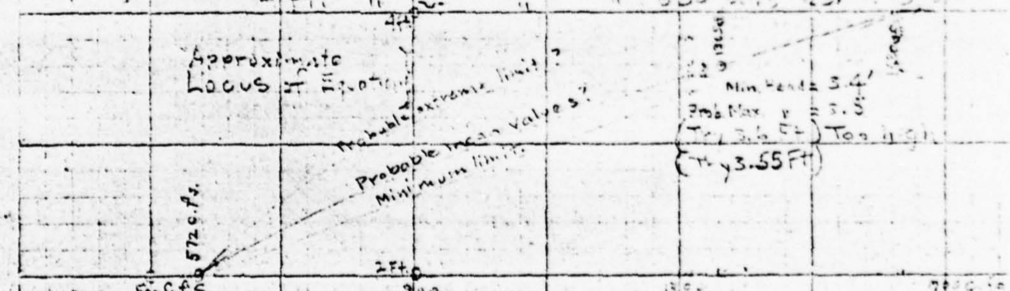
Total length, end wall to end wall = 60 ft.

" " each section in clear = 19.33± ft.

Determining approximate depth on crest

2 ft. depth ⇒ discharge of 572 c.f.s. (Engrs. Rept.)

4 ft. " ⇒ " " 1630 c.f.s. (Sht. 3)



b (For two end sections) $= 19.33 - \left(\frac{0.75 \times 3.6\pm}{10}\right) = 19.06'$ (Ref. Sht.)

b (For middle section) $= 19.33 - \left(\frac{1.5 \times 3.6}{10}\right) = 18.79'$

Approximate effective length $= 56.91'$

Discharge by E.B.C. curve $= 56.91' \times 24$ c.f.s./ft. $= 1365$ c.f.s.

Spillway Channel capacity with projecting stones in bottom (say 1 ft.) Using n = .03 on each side

$v = 60.67 \sqrt{3.33 \times .005} = 7.85$ ft/sec. } A = 5' × 20' = 100' r = 3.33 s = .005 (Sht. 1)

Q = 100 × 7.85 = 785 c.f.s. P = = 30'

CONT'D ON ACC.

1-23-17-3500 (16-3981)
Acc. 100
Form W96

STATE OF NEW YORK—CONSERVATION COMMISSION Dm427M

SUBJECT City of Johnstown FILE NO.
Dam 427 Mohawk (Serial No. 275) ACC. NO. C-3692
(See Sheet #1.) SHEET 16
COMPUTER [illegible] July 18, 1917 CHECKED BY 19
MADE IN CONNECTION WITH
REFERENCE CON'T FROM ACC. C-3691

Overfall Section - Investigation of Stability Cont'd
(Assumptions as Stated on Sheet 15)

Depth on Crest - Determination of Continued

Try 3.55

b_1 (For two end sections) $= 19.33 - \left(\frac{.75 \times 3.55}{10}\right) = 19.06 \pm$

b_1 (For middle section) $= 19.33 - \left(\frac{1.5 \times 3.55}{10}\right) = 18.80 \pm$

Approximate total effective crest length $= 56.92 \pm$

Resulting discharge by E.S. Cullings curve $= 23.5 \times 56.92 = 1338$ c.f.s.
This value plots on curve (Sheet 15) OK.
Reading from such curve 1314 c.f.s. $= 3.51 \pm$

3.51 ± of water on crest will therefore be used.

Resisting Moments:

	Area	Arm Ft.	Ft. Lbs
Mr (Masonry) in Ft. Lbs. (Sht. 5)			24,300 ′#
Water			
(B) Neglected			
(C) From Sht. 5			+1.5
(D) Prism on crest (Assumed as equal to ½ the section of prism 2.5 × 3.51′) =	4.39 □′		
Arm:		6.9	
Moment:			+30.25
(E) Prism of water sliding on apron neglected.			
$Mr_{(water)} = 31.75$ ft × cu.ft × 62.5 #/cu.ft.			= 1,983 ′#

Mr = Total Resisting Moment +26,283 ′#

Overturning Moments:

Static: (See Tables C-3581.)

Depth	Force	Moment about Base	Mom. about Top
10.51′	+3,451.90#	+12,093.36′#	+24,125.73′#
3.51′	−385.03#	−450.55′#	901.19′#
F	= 3,066.87#		23,285.54′#
F × 10.51′			32,250
$Mo_{(static)}$ =		8,964	8,964

CON'T ON ACC.

STATE OF NEW YORK—CONSERVATION COMMISSION Dm427M

SUBJECT ___ FILE NO. ___
ACC. NO. C-3693
SHEET 17
COMPUTER ___ 4-19-1917 CHECKED BY ___ 19__
MADE IN CONNECTION WITH
REFERENCE

Overall Section - Investigation of Stability - Continued.
(Assumptions as stated on sheet 15)

Overturning Moments - Continued

$M_{o(static)}$	Forward from Sht 16			8,964
Uplift: (Ref. C-3581 - Shts 65 & 83)				
Force: = $(656\ldots \times 7.7 \times \frac{2}{3})$ = 1690				
Arm:	(Sht 6)		5.13	
$M_{o(uplift)}$				8,660

M_o = Total Overturning Moment 17,624

$M_r - M_o$ = Net Resisting Moment (Sht 15) = 8,659

Vertical Component		Weight
Masonry (Sht. 6)	35.3 x 140	4,950
Water ① (Sht. 5)	area + 0.215	
ⓓ (Sht 16)	4.39 ... 4.61 cu.ft @ 62.5 =	288

Total downward pressure = + 5,238
Uplift as above = 1,690

V_E = 3,548

$U = \frac{M_r - M_o}{V_E} = 2.44$ (Arm of V_E; 5.1% outside middle third.)

$\frac{l}{3} = \frac{7.7}{3} = 2.57$

$F = \frac{V_E}{H_o} = \frac{3,548}{3,067}$ (Sht. 16) = 1.16 Coefficient of Stability

$f = \frac{H_o}{V_E}$ = .865 {Coefficient of friction should not exceed .70 or .75

$P_o = \frac{V_E}{l}\left(1 - \frac{6e}{l}\right) = \frac{3,548}{7.7}\left[1 - \frac{6(\frac{7.7}{2} - 2.44)}{7.7}\right] = 44.3$ #/□ or .3 #/□ Negligible

Neglecting uplift should be O.K. as on sheet 7.

Cont'd on Acc. C-...

1-23-17-2000 (10-2061)
Acc. 100
Form W86

STATE OF NEW YORK—CONSERVATION COMMISSION Dm 427 M

SUBJECT City of Johnstown FILE NO.

Dam 427 Mohawk (Serial No. 275) ACC. NO. C-3694
(See Sheet-1) SHEET 18

COMPUTER [illegible] 192[illegible] 19 17 CHECKED BY 19

MADE IN CONNECTION WITH

REFERENCE [illegible] Sht 10, Acc 3676 Con't from Acc. Sht 10 +C3[illegible]

Insert of computation for which there was insufficient space on Sheet 10 where result was embodied

(H_{es}) Resisting moment due to pressure of earth on back of dam.

Assumptions Involved:

(1) Weight of damp sand or loam 100#/cu.ft.
(2) Angle of repose of same material about 33°41'±
(3) Joints assumed in upstream paving slab.
(4) Load of concrete paving slab and 11-ft. of water superimposed upon prism of maximum pressure.
(5) Coefficient of friction of .35, between the damp sand or loam and back of dam.
(6) That the theory of earth pressure gives results sufficiently accurate.

Angle of maximum pressure:

$\frac{90° - 33°41'}{2} = \frac{56°19'}{2} = 28°9.5'±$

Tan. 28°9' = .535 = $\frac{x}{7'}$

x = 3.745

Superimposed Loads:

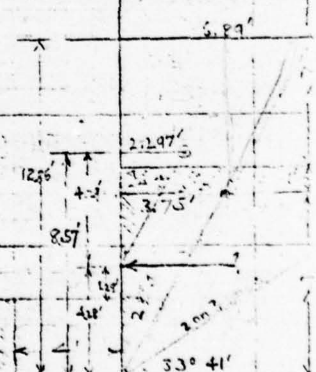
3.75' × 1' × 140#/cu.ft. = 526# Concrete

3.75' × 11' × 62.5#/cu.ft. = 2580# Water

Total Load 3,106#

Equivalent prism of Earth:

$\frac{3106^\#}{100^\#/ft^3}$ = 31.06 Cu.Ft.



Areas proportional to square of same dimension.

$\frac{\frac{1}{2} \times 3.75 \times 7}{31.06 + (\frac{1}{2} \times 3.75 \times 7)} = \frac{13.1}{44.16} = \frac{7^2}{y^2}$ $y^2 = \frac{49 \times 44.16}{13.1} = 165.5±$

y = 12.86± $\frac{12.86}{3} = 4.287'±$ 12.86'−4.287=8.57

$\frac{3.75'}{7'} = \frac{6.89'}{12.86'} = \frac{2.297'}{4.29'}$ $\frac{6.89'}{3} = 2.297'$ 8.57'−4.29'=4.28

Moment arm about base of dam = 4.28' − 3' = 1.28' (Used on Sht. 12)

Con't on Acc.

State of New York—Conservation Commission DM-427M

Subject: Dam 427 Mohawk (Series No. ...)

File No.

Acc. No. C-3695

Sheet 19

Computer ... 19... Checked by ... 19...

Made in connection with

Reference: ... on Sht. 10, Acc. 3677

Insert continued from sheet 18

Moments of Prism of Maximum Pressure

About back of section of dam

	Area	Arm	Moment
Rectangle:			
Area: 3.75' x 8.28' =	31.1		
Arm: 3.75 ÷ 2			
Mom:		=	58.2
Triangle:			
Area: $\frac{3.75}{2}$ x 7			
Arm: 3.75 ÷		1.25	
Mom		=	16.4
Totals	44.2		74.6
Resulting moment arm		1.69	

About base of section (wall)

	Area	Arm	Moment
Rectangle:			
Area:	31.1		
Arm: $\frac{8.28}{2}$ + 7		11.14	
Mom:			346
Triangle:			
Area:	13.1		
Arm: $\frac{1}{3}$ x 7		4.67	
Moment			61.2
Totals	44.2		407.2
Resulting moment arm:	9.22		

SUPERSEDED By data added on Sheet 18 Acc C-3644

Cont'd on Acc.

39-17-2500 (10-3081)
Acc. 106
Form W86

STATE OF NEW YORK—CONSERVATION COMMISSION DM 427M.

SUBJECT City of Johnstown FILE NO.
Dam 427 Mohawk (Serial No. 275) ACC. NO. C-3696
SHEET 20

COMPUTER 1917 CHECKED BY (Sheet #19 discussed) 19

MADE IN CONNECTION WITH

REFERENCE (H_{a} on Sht. 10, Acc 3696) CONT'D FROM ACC. C-3695

Insert continued from Sheet #19:

Point of application of resultant (Sketch on sht. 18)

Total Weight of Earth and Superimposed Loads:

$$3,106^{\#} + \left[\frac{7}{2} \times 3.745 \times 100\right] = 4,417^{\#}\pm$$

(1310.75# 137.25 374.5)

Horizontal Pressure of earth and superimposed load:

$$\frac{P}{W} = \frac{3.75}{7} = \frac{2,370^{\#}}{4,420^{\#}}$$

$$P = 2,370^{\#} \quad \text{(Used on Sht. 12)}$$

Frictional Resistance between earth and dam

Assuming coefficient of friction = .35 ±

$$f = \frac{F_m}{P} = .35 = \frac{830^{\#}}{2370^{\#}}$$

$F_m = 830^{\#}\pm$ Resistance (Used on Sht. 12.)

Insert of computation under title "Sliding" on sheet 14

Assuming coefficient of friction = .45

$$f_1 = \frac{H_o}{V_o} = .45 = \frac{5,915^{\#}}{13,132^{\#}} \therefore H_o - H_o = 9,006^{\#} - 5,915^{\#} = 3,091^{\#}$$

(See sheet 20) Showing resisting value of $\frac{1,031^{\#}}{2,060^{\#}}$

Determining whether earth would resist such force:

Exposed area = 3' × 1' ∴ Unit intensity = $\frac{2,060}{3}$ = 687±

Assuming coefficient of friction of wet loam = .50

The horizontal movement of 2 × 68.7 = 137.4 cu. ft. of subsoil would resist 687#

This small amount would undoubtedly be safely resisted by the pressure turned against the bottom of the dam by the cut off wall 4' thick, or by resistance of the earth below and at toe of the dam against sliding or bulging. Tan ∠ of repose = .668± = $\frac{1}{1.496}$ = $\frac{460^{\#}\text{ Vertical}}{687^{\#}\text{ Horizontal}}$

3 × 460# = 1380# Vertical

Point of application as shown

CONT'D ON ACC.

STATE OF NEW YORK—CONSERVATION COMMISSION Dm 427M

SUBJECT City of ... FILE NO. ...

Dam 427 Mohawk (Serial No. 275) ACC. NO. C-3697

SHEET 21

COMPUTER ... 19 7 CHECKED BY ... 19

MADE IN CONNECTION WITH

REFERENCE (Inserted computation to follow Sht. 14) CON'D FROM ACC. C-3...

Effect upon computed values of values used for Earth Pressures, Etc.

$M_{r(H+I)}$ = (As first computed) (Sht. 12) (H) +9,630'#

(" 10) (I) + 343'#

Total resisting moment of earth +9,973'#

$M_{o(H+I)}$ = (As first computed) (Sht. 12) (E) -3,033'#

(" 12) (F) 3,291 - 258'#

$M_{r(H+I)} - M_{o(H+I)}$ = 6,682'# Involved here...

Effect upon computed values of elements neglected.

Earth pressure upon down stream side of cut off wall with superimposed load.

$\frac{3.75'}{7'}$ (Ref. Sht. 13) = $\frac{Z}{3}$ Z = 1.61'

$\frac{L}{2} = \frac{11.6}{2} = 5.8'$ (Sht. 4) U = 6.37' (Sht. 13)

Prism of maximum pressure on down-stream side of cut off wall seems to lie entirely upstream from center of base. The position of the point of application of the resultant is such that the greatest portion of the load lies on the upstream side of the center of the base. It does not seem probable that the amount of the superimposed load will vary much so near the center (even under somewhat varying conditions) the loads already computed will therefore be used.

4.82' 9' 1.605' Bottom of dam 1.61' 3' 1'

5.99' 7.6' 11.6'

(Sht. H) 800#/□ 1470#/□ (Sht. 14)

1,146#/□ 1,239# 946 1,146# 433 1,239# 670

2,385#/□ ÷ 2 = 1,192.5#/□ Average intensity of pressure

Total superimposed load = 1,192.5#/□ × 1.61□ = 1,925# are 19.25□ of earth surface

CON'D ON ACC.

STATE OF NEW YORK—CONSERVATION COMMISSION

SUBJECT: Dam 427 Mohawk (Serial No. 275) FILE NO. ACC. NO. C-3698

SHEET 22

COMPUTER: [illegible] 22 July 1917 CHECKED BY 19

MADE IN CONNECTION WITH

REFERENCE: (Insert to follow Sht 14, continued) CONT'D FROM ACC. C-3697

Effect of Earth Pressure upon Computed Values, Continued

(Sketch on Sht. 21.)

Areas proportional to square of same dimension

$$\frac{\frac{3}{2}\times 1.61}{(\frac{3}{2}\times 1.61)+19.25^{\square}} = \frac{3^2}{y_1^2} \qquad y_1^2 = \frac{9\times 21.66}{2.415} = 80.8$$

(2.415; 21.665)

$$y_1 = 9^{\pm}$$

$$\frac{1.61'}{3} = \frac{4.815}{9} = \frac{1,160^{\#}}{2,167^{\#}} \qquad \frac{4.815}{3} = 1.605'$$

Loads:

(Sht. 21.)	Superimposed	= 1,925#
	Earth proper	= 242±#
	Total	= 2,167#
	Horizontal pressure	= 1,160±# (Ref– above.)
	Used on Sht. 14	129
	To resist sliding	1,031# (Used on Sht 20.)

Retaining Walls at ends of Overfall Section:

Assuming open joint at natural surface behind overfall section (El. 1047)

$P = 2,062.8$ lbs. (Ref– C 3589)

Investigating section only 1 ft. thick. (Omitting static pressure)

$M_o = 8,251.2$ (Ref– C 3589)

M_r { $4'\times 12'\times 2' = +96^{\square'}$; $(\frac{2}{2}\times 8')(\frac{4'}{3}) = -32^{\square'}$ }

$= 8,975 = 140^{\#} \times 64^{\square'}$

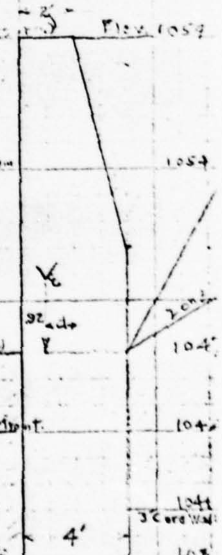
$M_r - M_o = 724'^{\#}$

$V_e = 785^{\#} = (48 + 8^{\square}) \times 140^{\#}$

$U = 0.92' \qquad \frac{l}{3} = \frac{4}{3} = 1.33'$

Point of application of resultant falls 30.8% outside middle ⅓ without considering static pressure (Surface behind)

$$P_o = \frac{V_e}{l} - \frac{6V_e d}{l^2} = \frac{785}{4} - \frac{6\times 785\times(2'-.92')}{16} = 122^{\#/\square'}$$



Note: While this wall stands 12' high for a horizontal distance of only 4-ft. still it must bear more or less static pressure in addition to earth pressure and its location is such that should a flood occur imediately after its failure, the upstream side of the embankment would be badly eroded which might result in the undermining of the core wall and failure of the entire structure.

CONT'D ON ACC.

STATE OF NEW YORK—CONSERVATION COMMISSION Dm 427M

Subject: Dam 427 Mohawk (Serial No. 270) — File No. — Acc. No. C-3699 — Sheet 23

Computer: 24, 1917 — Checked by: 19

Made in connection with:

Reference — Cont'd from Acc. C-3698

Overfall Section: Investigation of stability (Revision of data on Shts 9-14)

Same assumptions as stated on sheet 9 except that it is further assumed that static pressure acts only upon the upstream face of the dam and upon ½ the base of the cut off wall. This seems warranted if a row of suitable sheet piling is carefully driven to a good depth, and well bonded into the concrete for the entire width of the overfall section and well into the embankment at each end.

Resisting Moments:

$M_{r(masonry)}$	(Sht. 10)	104,700 #
$M_{r(water)}$	(" 10)	2,250 #
$M_{r(static)}$	(Combined with $M_{o(static)}$ below)	
$M_{r(earth)}$	[H(Sht.10)] + [L(Sht 22)] = (9,630 + 3,970)	13,600 #

M_r = 120,550 #

Overturning Moments:

(H) (Sht.12) → uplift under the base of dam — 3,033 #

(I) (Shts 21-2) Arm so short moment seems negligible — 0 0

(J+K) Static: (Ref-C3581) Total Head 19 Ft.

	Pressure	Mom. about Base wall		Mom. about W.S.
11 Ft.	11,281 #	71,448 #	× 2	= 142,896 #
4 Ft.	−500 #	666.7 #	× 2	= 1,334 #
			Diff	= 141,562 #
Lower 15 Ft	10,731 #	× 19′		= 204,600 #

63,038 # ÷ 10,731 # = 5.85 − 3 = 2.85 × 10,731 # = = 30,700 #

L (revised) Base effected, say 2 Ft.

1,187.5 #/□′ × 2 □′ × (11.6 − 10.6/2) — 25,200 #

M_o = 58,933 #

$M_r - M_o$ = 61,617 #

Vertical Components:

(K+I) (Shts 10 and 22) (830 # + 521.5) = 1352 #

Masonry: (Sht 13) + 13,380 #

Water: (Sht 13) + 326 #

+ 15,058 #

Uplift (above) 2 × 1187.5 # = − 2,375

V_E = Net 12,683 #

Cont'd on Acc.

29-17-3500 (10-7061)
co. 106
W95

STATE OF NEW YORK—CONSERVATION COMMISSION

SUBJECT City of Johnstown FILE NO. [illegible]
Dam #27 Mohawk (Serial No. 275) ACC. NO. C-3700
SHEET 24
COMPUTER Jy 24, 1917 CHECKED BY ____ 19__
MADE IN CONNECTION WITH
REFERENCE CONT'D FROM ACC. C-3699

Overfall Section – Investigation of Stability
Assumptions as stated on Sht. 23.

$U = \frac{M_r - M_o}{V_E} = 4.87' \quad \frac{L}{3} = \frac{11.6}{3} = 3.87'\pm$

Diff = 1.00

Point of application of resultant falls 26% within the middle $\frac{1}{3}$. According to the usual practice, the dam is therefore said to be safe against overturning about this plane.

Horizontal Components:

Static (Sheet. 23)	+10,781#
Earth (H_E Sht. 12)	+2,370#
	+13,151#
Earth (I Sht 22)	−1,160#
H_o =	11,991#

$F = \frac{V_E}{H_o} = 1.06\pm$ Coefficient of Stability very low.

$f = \frac{H_o}{V_E} = .945$ {Coefficient of friction should not exceed .50 to .60

$f_1 = .60 = \frac{H_{o1}}{V_E}$; $H_{o1} = 7,600$#

$H_o - H_{o1} = 4,391$# (This is

Note: Excess is due to considering static pressure as increasing below surface of ground (in true)

Resulting intensity of pressure per ft of length on down stream side of cut off wall = $\frac{4,391}{3}$ = 1,464±#/□'

This would cause an upward pressure on bottom of dam of about (Sht 20) 3,290#

$\frac{3'}{4.49'} = \frac{3,290^{\#}}{4,391^{\#}}$ Point of application unknown

$\frac{3290}{65} = 50.6$ {Cu. Ft. of material would resist it but probably not available.

CONT'D ON ACC. C-3770

STATE OF NEW YORK — CONSERVATION COMMISSION

SUBJECT: City of Johnstown (See Sheet #1) Ser# 275

FILE NO. Div 427M

ACC. NO. C-3770

SHEET 25

Filed by page No.

COMPUTER: [illegible] 27 Jy 1917 CHECKED BY

MADE IN CONNECTION WITH

REFERENCE

Con't from Acc. 3700

Overfall Section - Investigation of stability

Assuming horizontal joint at base of dam (El. 1042±)

Other assumptions as on Sht. 9 except that joint is ~~to be~~ assumed between cut off wall and base of dam

Crest overtopped to depth of only 3.51' as per flood by McKim formula (Ref - Sht. 16)

Material behind section has effective weight of only 100# - 62.5# = 37.5# per cubic foot = ω

Resisting Moments — Ft. Lbs. Sq.Ft x Ft

$M_{r\,(masonry,\ above\ El.\ 1047)}$ (Ref. Sht. 9) 634 × 140# = 88,750'#

$M_{r\,(water)}$

B neglected.

C (Sht. 5) 0.215 × [7.5 + (11.6 − 7.7)] × 62.5# = 153'#

D (Sht. 16) 4.39 × [[illegible] + (11.6 − 7.7)] × 62.5# = 2,850'#

$M_{r\,(earth)}$ (Sht. 26) .35 × 174.3# × 11.6' = 708'#

M_r = + 92,461'#

M_o = (From Sht. 26) − 45,468'#

$M_r - M_o$ = 46,993'#

Vertical Components:

Masonry	= 83.5 × 140#	= + 11,660# (Sht. 13)	
Water	(Sht. 17)	= + 288#	
		+	+ 11,948#
Uplift:	D (Sht. 11)	= − 1,022#	
	N (Sht. 12)	= − 588#	
			− 1,610#

V_E = 10,338#

U = $\frac{M_r - M_o}{\ }$ = 4.54' $\frac{L}{3}$ = 3.87±   Diff. 0.67   {Point of application of resultant falls 17.3% within middle third.

Con't on Acc.

1-23-17-2500 (10-2961)
Acc. 106
Form W86

STATE OF NEW YORK—CONSERVATION COMMISSION Dm 427M

SUBJECT City of Johnstown FILE NO.

Proposed Cech Center Cr. Dam ACC. NO. C-3769

(See Sheet #1) Ser # 275 SHEET 26

COMPUTER J. R. Henry 27 Jy 1917 CHECKED BY 19

MADE IN CONNECTION WITH

REFERENCE CONT'D FROM ACC. C 3770

Overfall Section - Investigation of stability

(Assumptions as on Sht. #25)

Overturning Moments

$M_{o(static)}$ (Ref - Sht. 16)

$\frac{8964'^{\#}}{3,067^{\#}} = 2.922'$ ft moment arm

$+ 5.0' = (8'-3')$

$7.922'$ Corrected arm. 24,300'#

$M_{o(Earth)}$

(H) Weight of superimposed slab $= 140^{\#} - 62.5^{\#} = 77.5^{\#}/\square = v$

$P = (\frac{1}{2}wh^2 + vh)\tan^2(45 \pm \frac{1}{2}\phi)$ (Ref. Am. C.E. Pkt. Bk p.583)

$= \left(\frac{37.5 \times 16}{2} + 310.0\right)\tan^2(45 - \frac{33°41'}{2}) = 174.3^{\#}$

$\text{Arm} = \frac{.333h(wh+3v)}{wh+2v} = \frac{1.33[(37.5\times4)+(3\times77.5)]}{(37.5\times4)+(2\times77.5)} = 1.67'$

$150.0 + 155.0 = 305$

291'#

(J) (Sht. 11) 7,963'#

(L) (Sht. 11) 9,914'#

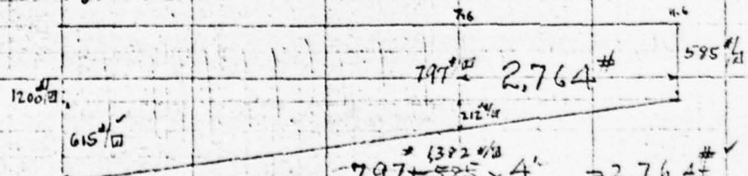
(N) (Sht. 12) 3,000'#

M_o = (Used on Sht. 25) 45,468'#

Subfoundation Loading

$P_a = \frac{V_e}{l}\left(1 + \frac{6e}{l}\right) = \frac{10,338}{11.6'}\left[1 + \left(\frac{6 \times 0.67}{11.6'}\right)\right] = 1,200^{\#}/\square$

$P_b = \quad = 895 \times 1 - .346 \quad = 585^{\#}/\square$



$\frac{797 + 595}{2} \times 4' = 2,764^{\#}$

$\frac{1200 + 797}{2} \times 7.6 = 7,574^{\#}$

CONT'D ON ACC.

22-17-2500 (10-2961)
ce. 100
W96

STATE OF NEW YORK — CONSERVATION COMMISSION Dm 427M

SUBJECT City of Johnstown FILE NO. [illegible]
Proposed Cork Center Cr. Dam ACC. NO. C-3768
(See Sheet 1.) Der #276 SHEET 27
COMPUTER [illegible] 29 Jy 1917 CHECKED BY Filed by [illegible]
MADE IN CONNECTION WITH
REFERENCE CONT'D FROM ACC. C-3769

Overfall Section — Investigation of Stability
(Assumptions as on Sht. 25)

Sliding Resistance (Ref - Sht 26)

Load on material of cut off wall = 2,764#
Coefficient of friction: concrete = .65
Resistance offered = 1,800#

Load on material of subfoundation = 7,574#
Coefficient of friction, concrete on wet clay = .35
Resistance offered = 2,650#

Total Resistance (INSUFFICIENT) = 4,450 #/ft.

Horizontal Components (Neglecting back water and apron slabs.)

Static (Sht. 16) 3,067#
" (Sht 11) $\frac{2}{3} \times 3{,}059$ 2,040#
Earth (Sht. 26) 174#

Total Sliding Force 5,281 #/ft.

Computed after making report. [illegible]

Unresisted according to above assumptions 831 #/ft.
(The movement of 17 cu. ft. of earth would probably resist this.)
If sand subsoil will take 10% of 7,574# more − 757

Yet Unresisted. PROBABLY SAFE = 74 #/ft.

REVISION Assume that cut off wall is to be built monolithic and well bonded with dam.

Earth pressure on back, revised (h = 7 ft.)

$$P = \left(\frac{wh^2}{2} + vh\right) \tan^2\left(45^\circ - \tfrac{1}{2}\phi\right) \quad \text{(Sht. 26)}$$

$$= \left(\frac{37.5 \times 49}{2} + 7 \times 77.5\right) \tan^2\left(45^\circ - \frac{33^\circ 41'}{2}\right) = 419\#$$

(919 × 24.5 + 542.5 = 1461.5; .256; 28°9.5' + 6°50.5')

Amount used heretofore = 174#

Increase (Forward to sht. 28) +245#

CONT'D ON ACC. [illegible]

1-23-17-3500 (10-2961)
Acc. 106
Form W85

STATE OF NEW YORK—CONSERVATION COMMISSION Dm 427M

SUBJECT City of Johnstown FILE NO. [illegible]
(See Sheet #1.) ACC. NO. C-3767
Ser # 275 SHEET 28

COMPUTER [illegible] Jy 27 1917 CHECKED BY .... 19..
MADE IN CONNECTION WITH
REFERENCE CONT'D FROM ACC. C-3768

Overfall Section – Investigation of Stability
(Assumptions as on shts. 25 and 27)

Sliding Resistance (Continued from Sht. 27)

(From sht 27) Forward +245#
(K) Static at back of cut off wall (Sht. 10) +980#

Total increase in horizontal component = +1,225#

Opposing Factors:

(G) ~~3'x4'~~ 1,680# x ~~140~~ #/□ x .35 = -588# 756

(P) 5.5± □ 772 x ~~140~~ #/□ x .35 = -270# 347

(I) $\frac{\Sigma V + 1,680 \; \textcircled{S} \; 12,018^{\#}}{11.6}$ = 1,037#/lin.ft {superimposed on subfoundation

Note: The use of the average load seems on the side of safety as the prism of maximum pressure lies entirely upstream from the center of the base. (Ref - Sht. 21) and the point of application of the vertical component of all forces was shown to intersect the base on the upstream side of the center (Ref - "U" on Sht. 13).

$P_p = \left(\frac{wh^2}{2} + vh\right) \tan^2(45^\circ \pm \frac{1}{2}\phi)$ (Ref sht. 26.)

$= \left(\frac{37.5 \times 9}{2} + 1,037 \times 3\right) .286 = 938\pm^{\#}$ = -938#

Sliding of section above base on wet clay = .35 x 10,338# (Ref - Sht. 25) = -3,620# 4655

Total resistance of active forces = 5,416#

Ho + 1,225# (Ref above and on Sht. 27) = 6,506#

Computed after making written report but I advised Mr. McK and Mr. Perkins of this result.

Unresisted by active forces = 1,090 #/ft longitudinal

Note: Since coefficient of friction of only .35 has been used above, and sliding on Wet loam with coefficient of .50 is now to be assumed, the difference, or .15 x 7574# may be deducted before considering quantity of earth which must be moved before failure would occur. (Ref - Sht. 27.)

.15 x 7,574 = [illegible] = 1,131#

N.B. Dam therefore seems SAFE AGAINST SLIDING if cut off wall has not been [illegible] (?)

Cont'd on Acc. C-3765

1-23-17-2500 (16-2061)
Acc. 106
Form W86

STATE OF NEW YORK—CONSERVATION COMMISSION

SUBJECT: City of Johnstown FILE NO. DS-...
Proposed Cork Center Cr. Dam ACC. NO. C-3765
(See Sheet 1) Ser# 275 SHEET 29
COMPUTER: [illegible] 28 Jy 1917 CHECKED BY
MADE IN CONNECTION WITH
REFERENCE

Overfall Section – Investigation of Stability
(Assumptions as on Shts. 25 and 27.)

Cut Off Wall – Stresses

Possible tension due to earth friction
Coefficient may equal .60

$H' = 245^{\#}$ (Sht. 27) x .60 = $147.0^{\#}$
$V' = 1,037^{\#}$ (" 28) x .60 = $622.2^{\#}$

1,037#

2,764# Sht. 27

147#

1.47'

I (Ref.– Am. C.E. Pkt. Bk. p. 583)

$$Arm = \frac{.33h(wh+3v)}{wh+2v}$$

$$= 1\frac{\cancel{375\times3} + 3\times1{,}037^{\#}}{\cancel{375\times3} + 2\times1{,}037^{\#}} = \frac{1{,}125 + 3{,}111 = 3{,}223.5}{1{,}125 + 2{,}074 = 2{,}186.5} = 1.47'$$

I

L

Note: Probably not worth while to analyze so many forces

The wall is employed merely to resist sliding of the section

Suppose (as an extreme case) that all such resistance were to be exerted as a tangential force about its base, then – (Ref Sht 27)

(?) $4,450^{\#} \times 3 \times 12'' = 160,000''^{\#} = \frac{bd^2 f}{6} = \frac{12'' \times 48 \times 48}{6} f$ (36", 2,304)

$$f = \frac{160,000''^{\#}}{4,608} = 34.7\ ^{\#}/\square$$ Roughly Approximate only.

Conclusions:

It would seem reasonable to require that such cut off wall be reinforced with steel to carry all tension

... 3/8" bars $4\frac{3}{4}$" ctrs would seem O.K. Roughly approximate only.

$$\frac{60 \times 12''}{4\frac{3}{4}} = 180$$

(16 15669)
Acc. 106

SUBJECT City of Johnstown, FILE NO. Dr. 4 [illegible]
Proposed Cork Center Creek Dam ACC. NO. M5590
Ser. #275 SHEET 30
COMPUTER [illegible] Dec. 31, 1918 CHECKED BY 19
MADE IN CONNECTION WITH Dec. 23, '18 and Dec. 13, '18 letters by Jas. P. Wells, Engr. and Dec. [illegible]
REFERENCE report of inspection by our Mr. McKim, as instructed by Div. Engr. CONT'D FROM ACC. C-3765

Equalization Effect of Reservoir as left when work was stopped:

Capacity of two 24" outlet pipes:

Note: The plans, (Acc. 8058) show but one such 24" pipe with outlet end far below the dam, and at end of the spillway channel, with elevation about 998±

The letters and reports do not state whether the two 24" pipes have their outlets at the same elevation, or at what elevation their ends have been left open

Assumptions:

① Gross head 20 ft. as indicated by Mr. McKim's report;

② That the ends of both pipes are at the same elevation, and that the effective head upon both will be the same;

③ That length of such pipes is about 230 ft. which is the approximate base width of the ~~[illegible]~~ embankment as shown by the plans (Acc. 8058) for the elevation at which such pipes appear to rest;

④ That loss of head from reservoir to inside of pipes = $.75\frac{v^2}{2g}$

Limits:

Maximum:

$$Q_1 = 2(AV_1) = 2(A\sqrt{2gh}) = 2(8.02 A\sqrt{h})$$

$$= 8.02(2 \times \overset{6.28}{3.1416}) \times 4.46$$

$$= 225 \pm \text{ c.f.s.} = 79.5 \text{ c.f.s./sq. mi.}$$

(Watershed = 2.26 sq. mi.)

$$v_1 = \frac{Q_1}{2A} = \frac{225}{6.28} = 35.9 \text{ ft/s.}$$ (Neglecting losses of head due to entrance, friction etc.)

Try Chezy formula and a value of .011 for "n." (Ref-Gr[illegible])

$$r = \frac{A}{P} = \frac{3.142}{6.283} = 0.5$$

$$S = \frac{20}{230} = .087$$

$$C_2 = 127.6$$

$$v_2 = C_2\sqrt{r_2 S_2} = 127.6\sqrt{.0435} = 26.6 \text{ ft.}$$

$Q_2 = 2(A V_2) = 6.28 \times 26$ [illegible]

CONT'D ON ACC. M [illegible]

22.8
133.4

STATE OF NEW YORK—CONSERVATION COMMISSION

Form W-86

SUBJECT City of Johnstown, N.Y. FILE NO. Dm-1-27M

Proposed Cork Center Cr. Dam ACC. NO. M5591

Ser #275 SHEET 31

COMPUTER J.W.H. Jan 3, 1919. CHECKED BY 19

MADE IN CONNECTION WITH See Sht. 30.

REFERENCE

CONT'D FROM ACC. M5590

Equalization Effect of Reservoir as work was stopped:

Capacity of Outlet works, Contd.

(Assumptions as on Sht. 30)

Limits, Continued:

Minimum:

Try Chezy formula and a value of .015 for "n" (Ref. C1750)

$v_3 = C\sqrt{rs} = 86 \times .208+ \quad = 17.9+ \text{ ft/sec.}$

$Q_3 = 2(Av_3) = 6.28^{\square'} \times 17.9^{f/s.} \quad = 112.5 \pm \text{ c.f.s.} \simeq 39.3 \text{ c.f.s./sq.mi.}$

Try $v_4 = 23.3^{ft/sec.}$

$\frac{.75v^2}{2g} = \frac{.75 \times 543^{\pm}}{64.32} = 6.33'$ {Loss at entrance or in rack, Assumption #4.

$.5Q_4 = Av_4 = 3.142^{\square'} \times 23.3^{f/s} = 73.2 \pm \text{ c.f.s.}$

Say C = 130 for Hazen Williams formula

$\frac{73.2^{c.f.s.}}{1.3} = 56.4 \text{ c.f.s.}$

$\simeq 36.4$ M.G.D.

$5.7^{ft/100'}$ due to velocity and pipe friction (R-1312)

or $13.1^{ft.}$ loss of head in 230' by assumption #3.

$13.1' + 6.3' \quad = 19.4 \pm$ ft. {Total head loss to discharge 73.2 f/s.

Therefore one 24" pipe would discharge 75± c.f.

The two would therefore discharge 150± c.f.s.

or the equivalent of 52.5± c.f.s/Sq.Mi.

provided that the rack and pipe can be kept flowing free and it is not clear by what means the racks are to be cleaned.

CONT'D ON ACC. M-5592

STATE OF NEW YORK—CONSERVATION COMMISSION

Form W-46

SUBJECT: City of Johnstown, N.Y., Proposed Cork Center Cr. Dam Ser #275 — FILE NO. [illegible] — ACC. NO. M5592 — SHEET 32

COMPUTER: A.E. Berhoust + M.J. Duffy 19__ CHECKED BY: A.E. Berhoust + M.J. [illegible] 19__

MADE IN CONNECTION WITH: See Sht. 30

REFERENCE: — CONT'D FROM ACC. M5591

Equilazation Effect of Reservoir (with dam completed to Elev. 1043

Area + Capacity Curve Data. (Used on Acc. M5597)

Planimetered Areas from Acc. 3056

							Mean	Sq. Ft.
Elev. 1031								
40072			38300	8595				
31400	8672	8660	30205		8590		5,625	271,000
22752	8648		21619	8586				
Elev. 1036								
37220	11649		21495	11583				
25571	11659	11654	9912		11573		11,613	365,000
13912			48349	11563				
Elev. 1041								
48163	17108		49479	17242				
31055	17160		32237	17228				
13895			15009					
		17154			17215		17,185	539,000
41981	17171		7615	17181				
24810			40434	17210				
			23224					
24791	17176							
7615								
Elev. 1046								
4410	34240		13734	34347				
29170	34181	34210	29237	34272	34310		34,260	1,076,000
35939			45115					
Elev. 1051								
38560	48858		22775	48850				
39702		48875	23925	47850	47850		47,862	1,532,000
			25075					
39625	48893							
40732								
Elev. 1054								
33179	58574							
24605	58585	58580	16020	58403				
33983			7617					
					58408		58,494	1,835,000
			7643	58413				
			49230					

Planimeter tested on Plat 500'x600' Area = 300000 sq. ft. 1 Plan. Unit = 300000 ÷ 9556 = 31.4 sq. ft.

24922	9562	9558	45650	9552	9554	9556
15360	9555		36098	9556		
5805			26532			

CONT'D ON ACC. M5597

Form W-88
6-19-19-3000
(16 15669)
Acc. 100

STATE OF NEW YORK CONSERVATION

SUBJECT City of Johnstown, N.Y., — FILE NO. Dm 427[illegible]

Proposed Cork Center Cr. Dam — ACC. NO. M5597

Ser. # 275 — SHEET 33

COMPUTER J.W.H. Jan. 24, 1919 CHECKED BY ______ 19__

MADE IN CONNECTION WITH See Sht. 30

REFERENCE — CONT'D FROM ACC. M 5592

Equalization Effect of Reservoir as work was stopped.

Area & Capacity Curve Data – Continued:

(Data from Acc. M5592)

	Areas	Sum	Sum/2	Interval ft.	Volume Increments	Volume – Integrated cu. ft.
El. 1022.5	43M					0
		314M	157M	7.5	1,385M	
" 1031	271 M					1,385,000
		636	318	5.0	1,590M	
" 1036	365 M					2,975,000
		904	452	5.0	2,260M	
" 1041	539 M					5,235,000
		1615	807.5	5.0	4,040M	
" 1046	1,076 M					9,275,000
		2,608	1,304	5.0	6,520M	
" 1051	1,532 M					15,795,000
		3,367	1,683.5	3.0	5,050M	
" 1054	1,835 M					20,845,000
	5,661	14 / 1,378 / 11,322 / 5,661	4,722.0	[illegible]	20,545,000	

– Plotted on Acc. 5[illegible]

CONT'D ON ACC. [illegible]

STATE OF NEW YORK—CONSERVATION COMMISSION

SUBJECT City of Johnstown, N.Y.,
Proposed Cork Center Cr. Dam
Ser. # 275

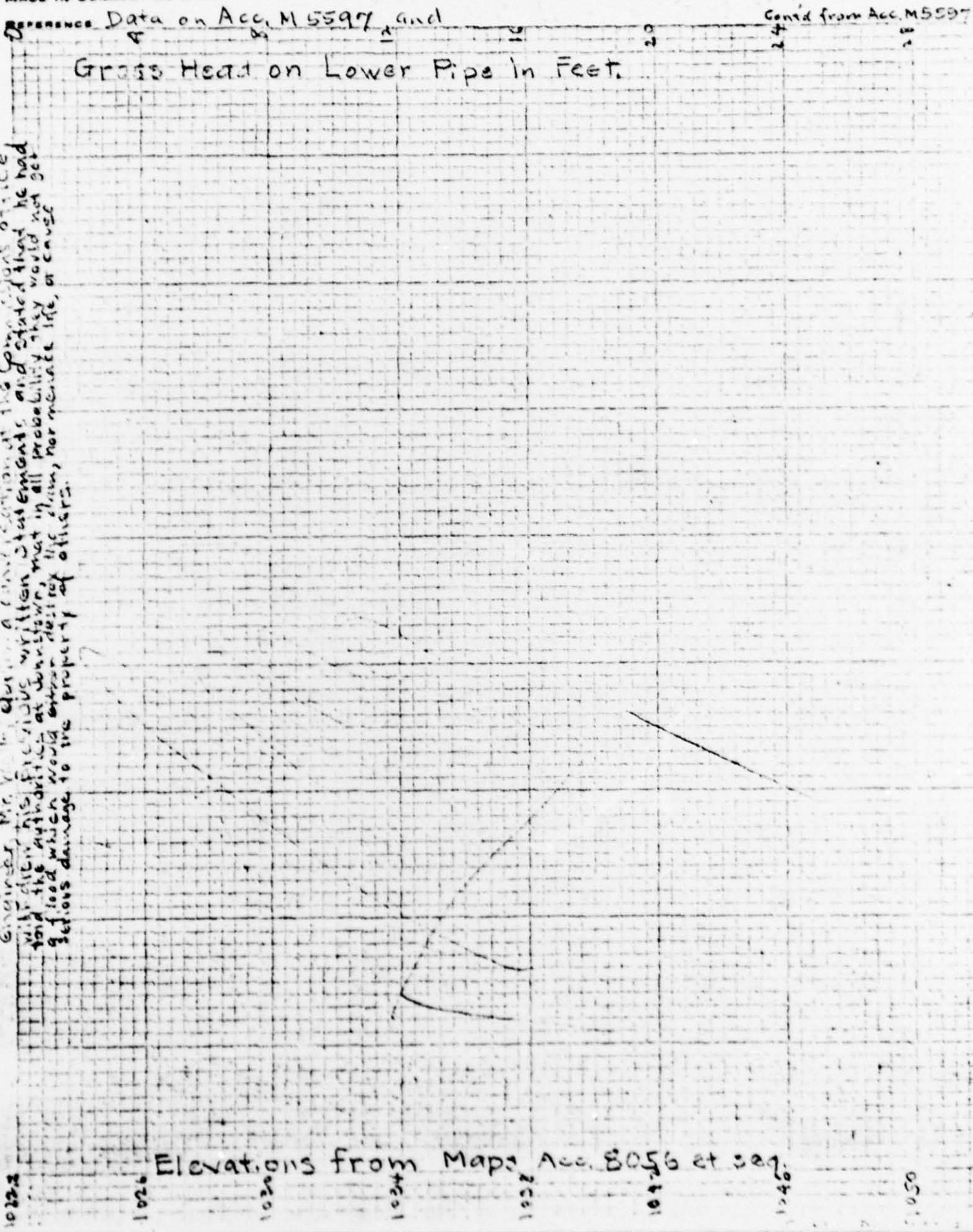
FILE NO. Dm 427M
ACC. NO. 3085
SHEET 34

COMPUTER [illegible] Jan 27 19 19 CHECKED BY 19

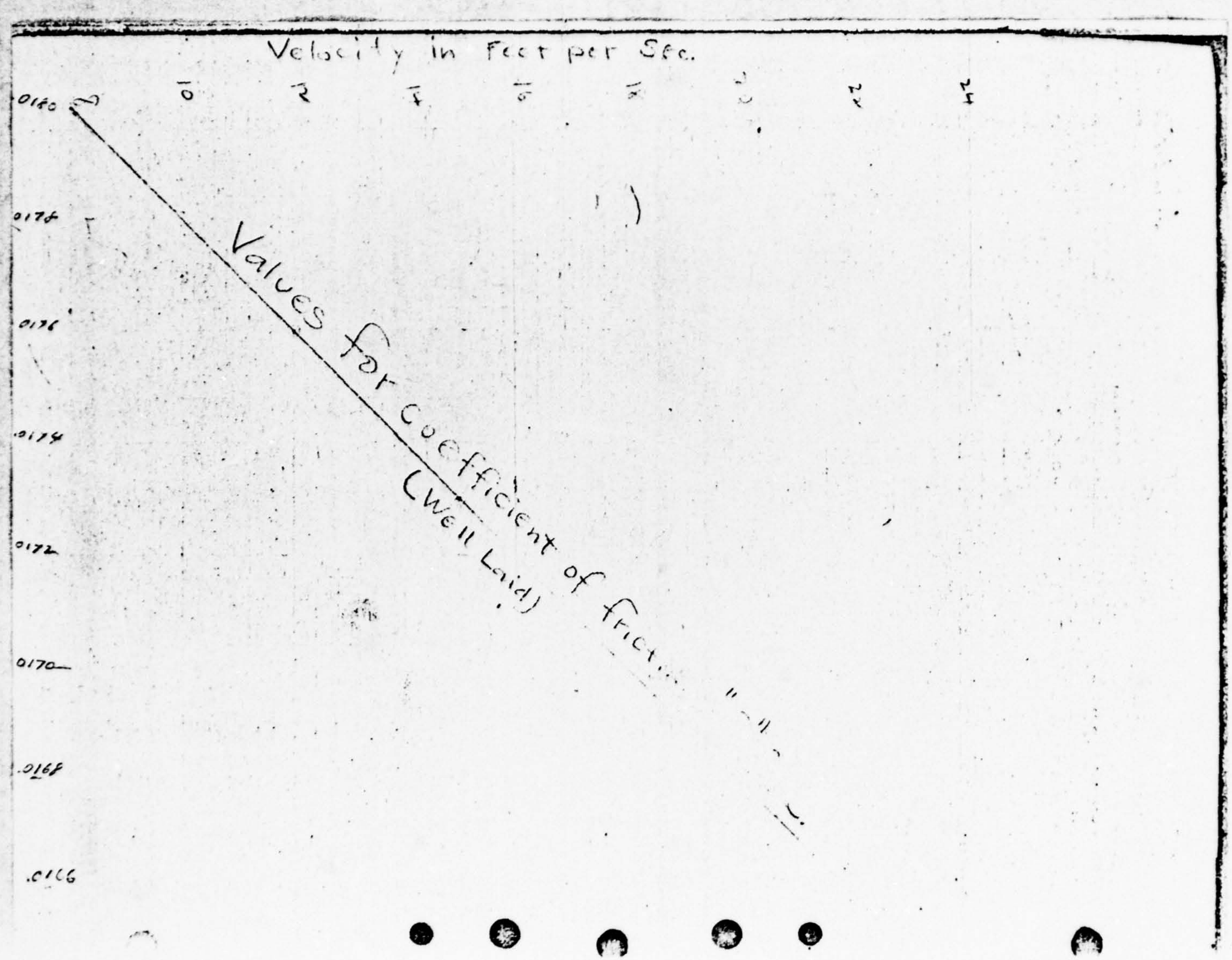
MADE IN CONNECTION WITH See Sht. 30

REFERENCE Data on Acc. M 5597 and

Cont'd from Acc. M5597



Computation [illegible] not continued as the applicant's Engineer, Mr. [illegible], during a conversation at the Commission's office with him, withdrew his previous written statements and stated that he had told the authorities at Johnstown, that in all probability they would not get a flood which would either destroy the dam, nor menace life, or cause serious damage to the property of others.



Velocity in Feet per Sec.
8
10
12
14
16
18
20
22
24
0180
0178
0176
0174
0172
0170
.0168
.0166
Values for coefficient of
(Well Laid)

12-8-14-2000 (16-2172)

Acc. 390
Form IW50

STATE OF NEW YORK—CONSERVATION COMMISSION

SUBJECT City of Johnstown, N.Y., Proposed Cork Center Creek dam. Ser.# 275

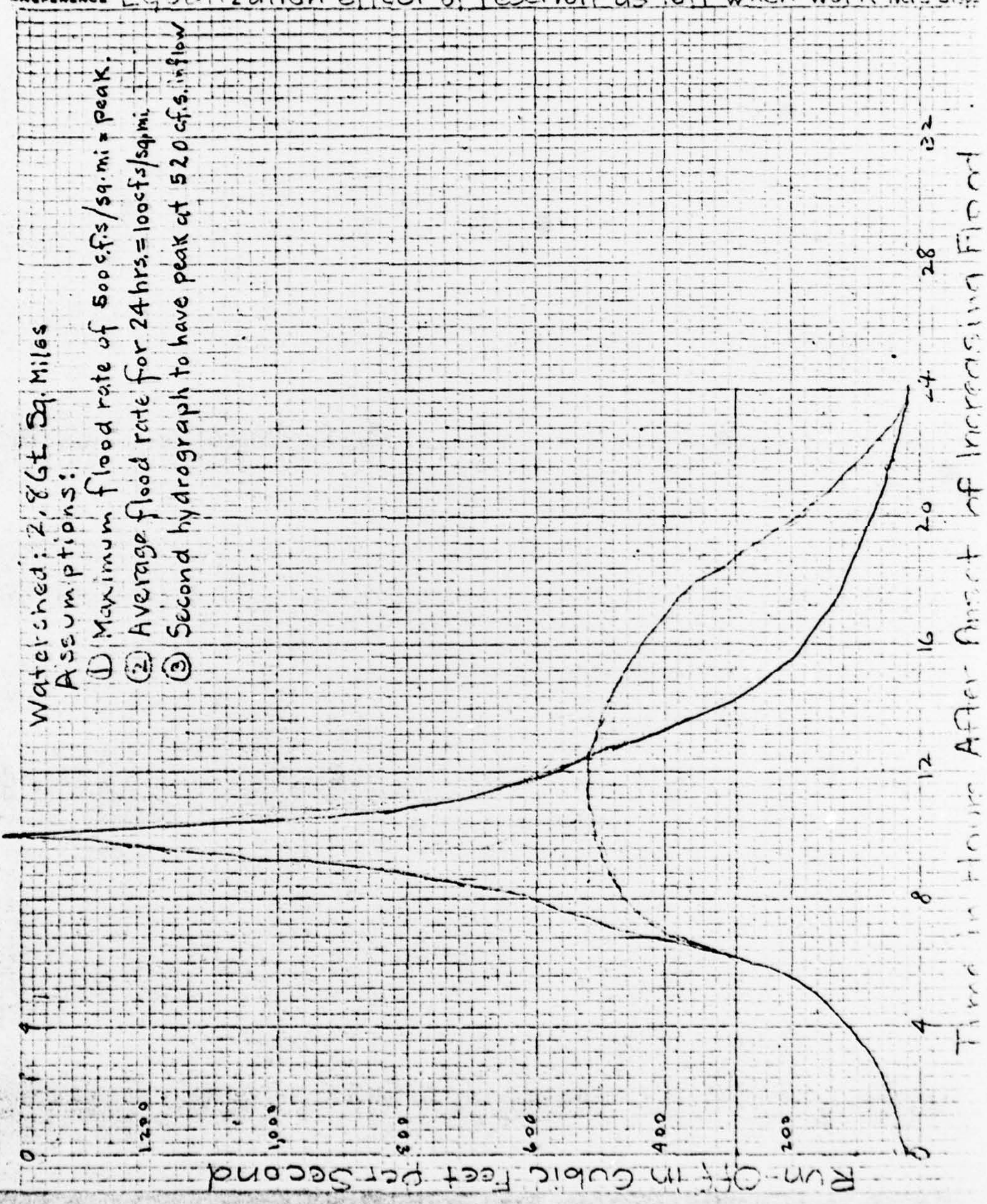
FILE NO. Dm 427 M
ACC. NO. M-4234
SHEET 35

COMPUTER J.E.H. Jan 24, 1919. CHECKED BY 19

MADE IN CONNECTION WITH See Sheet 30

REFERENCE Equalization effect of reservoir as left when work was stopp.

Cont'd from Acc. 308



150

EOR D. PRATT, COMMISSIONER
LEXANDER MACDONALD, DEPUTY COMMISSIONER
S. UGHTON, SECRETARY
AR ALL MCLEAN, DEPUTY ATTORNEY GENERAL

IN REPLYING PLEASE REFER TO FILE NUMBER

STATE OF NEW YORK

CONSERVATION COMMISSION

ALBANY

DIVISION OF FISH AND GAME
LLEWELLYN LEGGE, CHIEF
DIVISION OF LANDS AND FORESTS
C. R. PETTIS, SUPERINTENDENT
DIVISION OF WATERS
A. H. PERKINS, DIVISION ENGINEER
DIVISION OF SARATOGA SPRINGS
J. G. JONES, SUPERINTENDENT
SARATOGA SPRINGS, N. Y.

July 23, 1917.

Mr. A. H. Perkins, Division Engineer,
Conservation Commission,
P R E S E N T.

Dear Sir:-

Concerning the reservoir #427 Mohawk Watershed for the city of Johnstown (Serial #275):-

On July 20 I inspected the vicinity of the site for this dam. The soil I found to be a loamy earth with some bolders which were mostly of granite from a glacial formation. I found no outcropping of any ledge rock and the excavations will have to be carefully watched to see if there is any such.

Respectfully yours,

Alex Rice McKim

INSPECTOR OF DOCKS AND DAMS.

MvK:MH.

150

Nov. 6, 1916.

Mr. W. E. Natanson, City Engineer,

Johnstown, N. Y.

Dear Sir:-

We are informed that you are going to build a dam and desire information about the jurisdiction of the Conservation Commission over such construction. We, therefore, hand you herewith application blanks and instruction sheet.

Yours truly,

GEORGE D. PRATT, Comm'r.

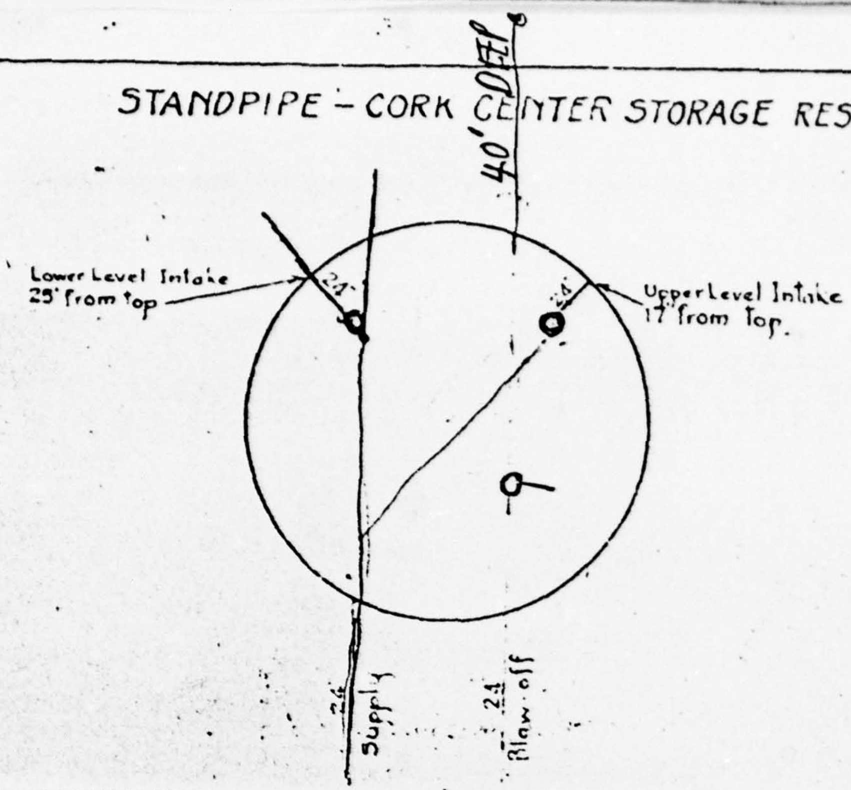
By, aHP

Division Engineer.

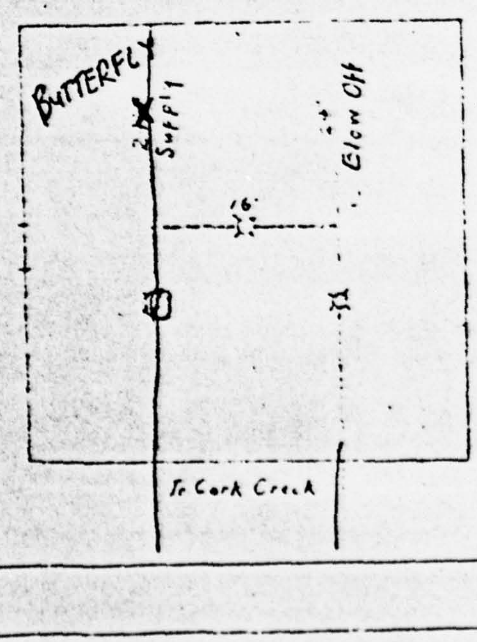
AHP/F

STANDPIPE – CORK CENTER STORAGE RESERVOIR

69



70



–GATE HOUSE–
CORK CENTER
STORAGE RESERVOIR

INSTRUCTIONS FOR STATE OF EMERGENCY CONDITIONS

1. Notify Abel and McGregor

Cold Brook

1.) Open gate in creek culvert under highway if you can get to it.

2.) Open mud gate in manhole between reservoir and road.

3.) Open hydrants, Johnson, Hall, Akin, John, Wells, Burton, Pearl Oak. Close down gate on Cayadutta Street at Main.

Storage

Open gate.

Open Mud Gate

Pray

Distributing.

1.) Open Bypass gate at Aeration Falls but don't let the channel overflow, especially down by Chlorination House.

2.) Open Mud Gate in gate house.

3.) No water should escape over any part of the dam except the concrete spillway.

RESERVOIR INSTRUCTION

Cold Brook

18" Below sidewalk - turn down gate in cornfield
Even with sidewalk - turn out intakes
12" over sidewalk - State of Emergency Exists, - call Skinny, Jr. & WM.H. McGregor

Storage

1.) When full, keep it from running over by opening gate. Keep it 2" - 6" down from edge.

2.) If not full, operation of gate depends on distributing reservoir operation.

Distributing

1.) When storage is not full, do not let intake run over spillway. Intake should range 6" - 42" below spillway level. Adjust by storage gate.

2.) When storage is full, intake can run over, but watch depth over spillway and fullness in concrete bypass channel.

3.) If any water begins to run over the grassy, earthen dam or around the sides of the spillway, a State of Emergency Exists.